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MAJ. GEN. HENRY WASHINGTON BENHAM
CADET U. S. M. A., 1833-1837
BORN 1813—DIED 1884

SEE PAGE 110

Action of Water in Locks of the Panama Canal.

BY

Col. H. F. HODGES,
Corps of Engineers, U. S. Army,
Engineer of Maintenance, The Panama Canal.

Ordinarily the operation of filling or emptying a small lock is as simple in practice as it is in theory, and the action of the water develops nothing unexpected. When the upper valves are opened to fill the lock, the water flows through them and through the culverts into the lock chamber with a velocity dependent upon the difference in the levels of the water in the upper pool and in the chamber. As the water level in the lock rises, this velocity becomes less and less, the flow finally ceasing altogether when the levels equalize. The last few inches toward equalization are provokingly slow in disappearing, and may take more time than the first few feet. It is not unusual to put a strain on the gate maneuvering gear before equalization has taken place, and to open the gates against a slight residual head, which seems likely to remain indefinitely.

The time spent in changing levels is given by the formula:

For single lift locks,

$$t = \frac{2A\ (\sqrt{H} - \sqrt{h})}{ca\sqrt{2g}}$$

For consecutive locks in a flight,

$$t = \frac{2A\ A^1}{A + A^1}\ \frac{(\sqrt{H} - \sqrt{h})}{ca\sqrt{2g}}$$

in which H is the original difference in level of the pools to be equalized, h is the final difference in level, being zero at actual equalization, A and A^1 are the areas of the locks in horizontal plan, a is the sectional area of the culvert or valve through which the water flows during the operation, g is the acceleration due to gravity; and c is a coefficient upon which much depends. The formula

for single lift locks applies also to filling the upper lock of a flight and to emptying the lower one. The formula for consecutive locks of a flight applies where such locks equalize levels with each other.

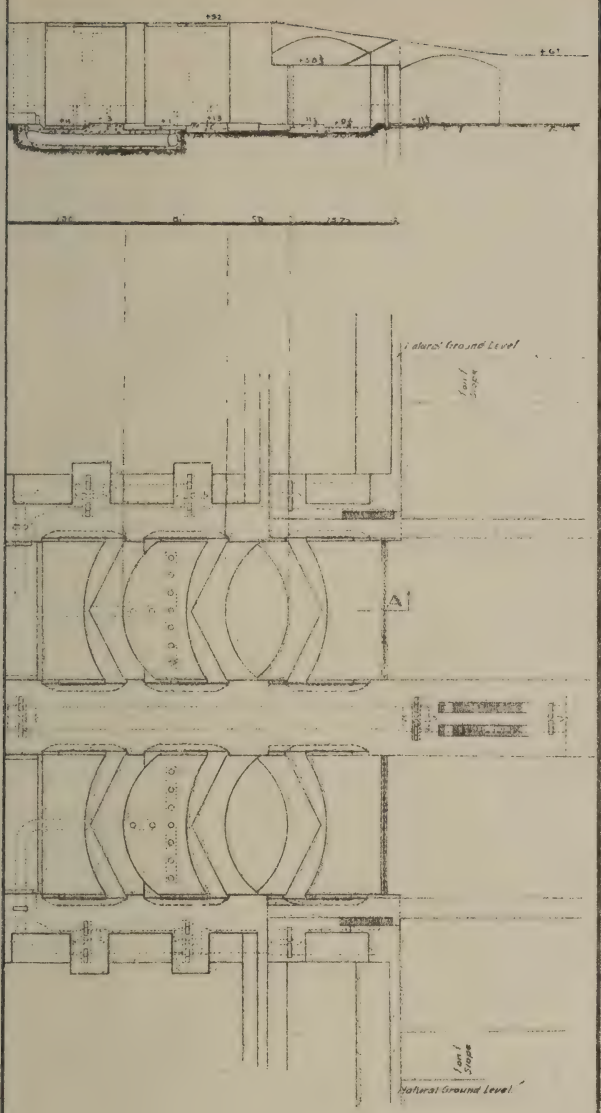
The arrangement of the culverts and valves is the same in principle in all the locks of the Panama Canal; and may be illustrated by the Pedro Miguel single lift lock. (Figs. 1 and 2.)

The water in either of the twin locks may be admitted or released through two culverts, one at the base of the side wall, communicating freely with the lock chamber, the other near the bottom of the middle wall, communicating through valves with the lock chambers on either side. The communication of the main culverts with the lock chamber is through lateral culverts running under the lock floor at right angles to the walls, and having, each, five openings leading upward into the chamber. The main culverts are regulated by Stoney valves, two sets being provided near each end for safety, to separate the chamber from the upper and lower pools. The side culvert has also another set near the middle gates. The laterals from the middle culvert are closed by cylindrical valves, to allow communication from that culvert with either lock to be established or interrupted at will. The laterals from the side culverts, which feed only one lock chamber, need no individual valves.

By means of the intermediate gates, the chamber may be separated into two; one, say, 600 feet, and the other, say, 400 feet in length. By using the entire chamber below the upper guard gates, a length of 1,000 feet may be made available. Below the upper operating gates the length is 900 feet. Let us consider the 900-foot lock, extending from the upper operating gates to the fender chain protecting the lower safety gates.

900-Foot Lock.

The side culverts draw from the upper pool through three openings, each closed by a Stoney valve. These valves are called guard valves, but may be used for operation, and form one of the two sets separating the chamber from the upper pool. Each valve closes an opening 8 by 18 feet = 144 square feet in area. The second set of filling valves lies 320 feet below the entrance and consists of two of the same size as the guard valves, each closing an opening 8 by 18 feet = 144 square feet in area. Between the two sets of valves, the culvert has a sectional area of 333 square feet. Below the lower set of filling valves, it has an area of 255 square feet. It communicates with the lock chamber through eleven lateral cul-



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PEDRO MIGUEL LOCKS
GENERAL PLAN AND SECTIONS

Fig. 1.

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PEDRO MIGUEL LOCKS
GENERAL PLAN AND SECTIONS

Fig. 1.

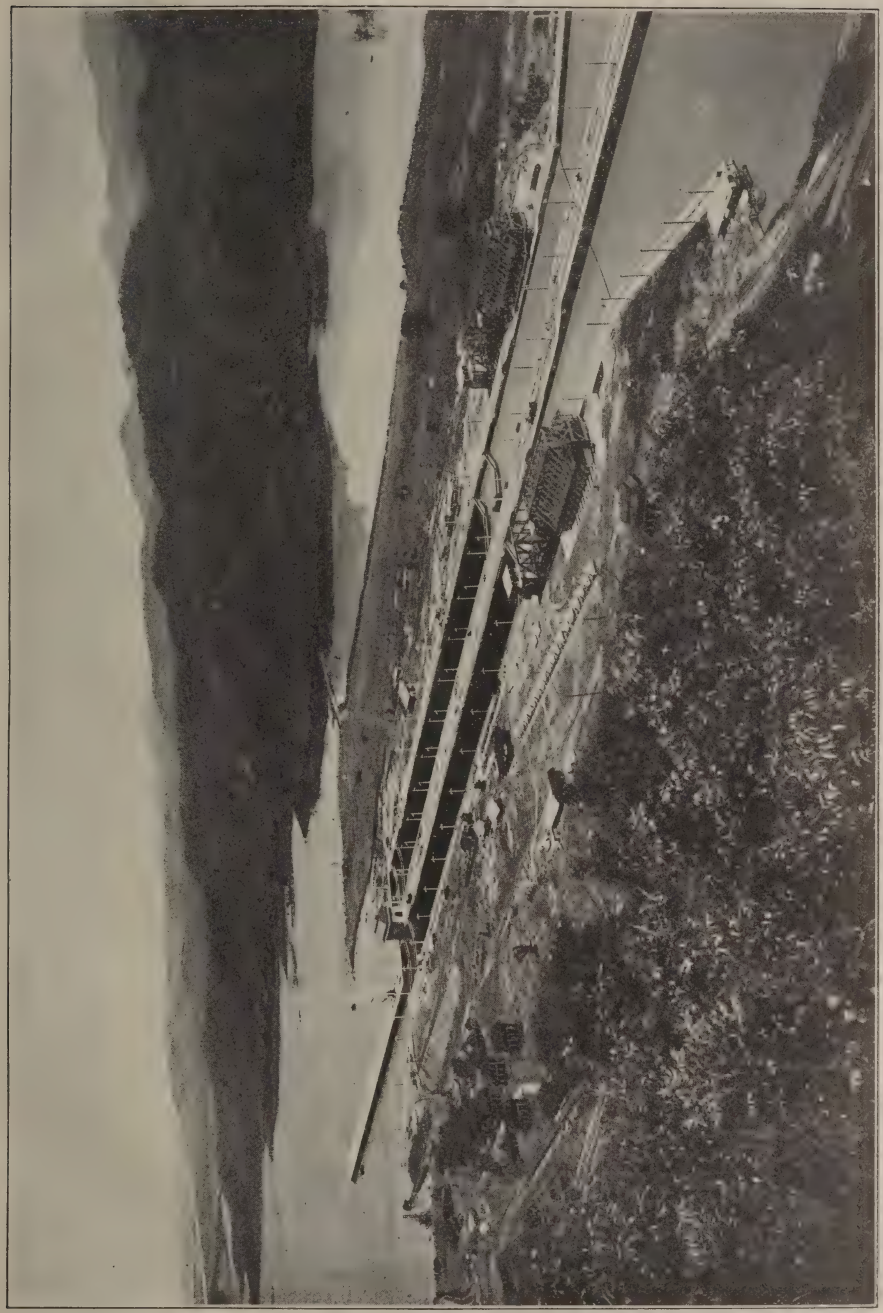


Fig. 2. General View of Pedro Miguel Lock from Luisa Hill.

verts, each of which joins the main culvert through an opening having a sectional area of 55 square feet. The combined area of the openings is therefore 605 square feet. The laterals have each a cross section of 41 square feet, or a combined area of 451 square feet. Each lateral opens into the lock chamber through five circular apertures, each 12.23 square feet in area. The combined area of these openings is therefore 61.2 square feet for each lateral, or 673 square feet for the system. The course from the lake to the chamber, shown diagrammatically in Fig. 3, lies through (a) guard valves, 432 square feet; (b) large culvert, 333 square feet; (c) filling valves, 288 square feet; (d) main culvert, 255 square feet; (e) throats of laterals, 605 square feet; (f) laterals, 451 square feet, and (g) openings into chamber, 673 square feet. It is most contracted at (d) the main culvert, 255 square feet in area, provided that both the filling valves (c) are raised.

In the middle culvert, the condition is similar except that the two sets of filling valves are exactly the same, and the large culvert extends from the entrance to the upper pair of filling valves. The entrance may be taken at 360 square feet in area. There are but ten laterals, and the throat of each has an area of 33.2 square feet. The waterway has, therefore, successive cross sections of 360 square feet, 333 square feet, 288 square feet, 255 square feet, 288 square feet, 332 square feet, 410 square feet, and 612 square feet. It is most contracted, as before, in the main culvert with area of 255 square feet.

In designing the water supply system of the locks, the sectional area of the main culvert was first fixed at a value which, with ordinarily good flow, would fill the lock in a reasonable time. The value of the coefficient c was assumed at 0.65, which is more favorable than has been found experimentally at most locks. It was thought safe to take this favorable value on account of the large size of the culverts, and the care which was taken to round the edges of the intakes. With this assumption, and with normal pool levels above and below, the side culvert alone would fill or empty the Pedro Miguel 900-foot lock in 16 minutes and 25 seconds of actual full flow. It was assumed that, by using the middle culvert also, this time could be materially reduced; and that it would be safe to count on completing the operation of filling or emptying the lock with both culverts in fifteen minutes, including the time taken to open the valves. In computing the time of equalization, the area a, through which the stream flows, was taken at the most con-

tracted point: for the 1,000-foot, 900-foot, or 600-foot locks, this would be at the main culvert, when both of the valves are fully open.

FILLING LOCK, SIDE CULVERT ONLY.

Since the locks have been in service there has been frequent

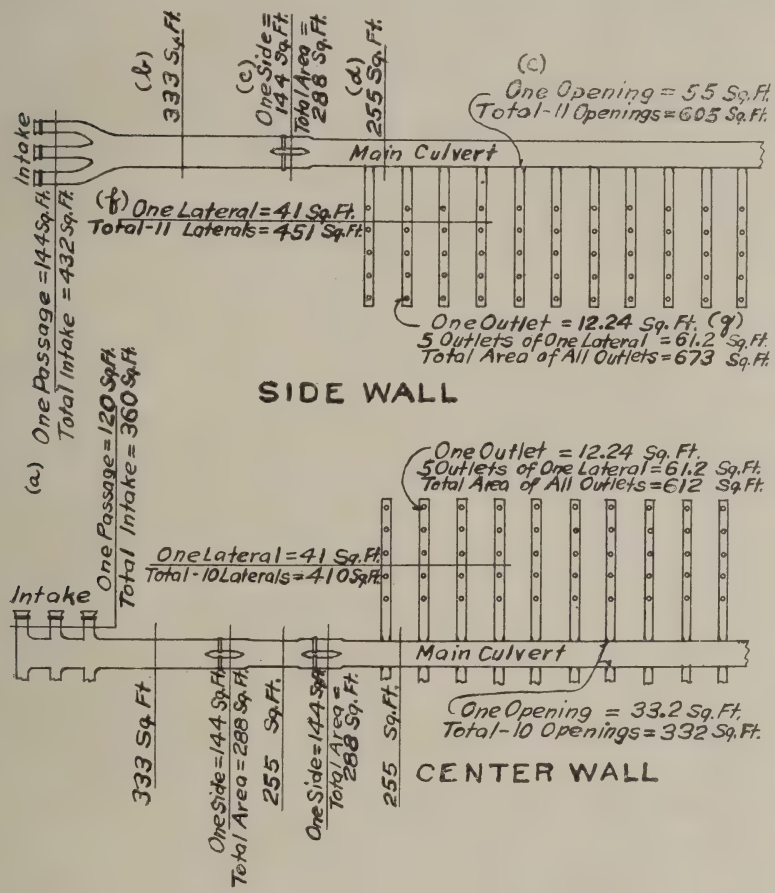

(b) 333 Sq.Ft.
(c) One Side = 144 Sq.Ft. Total Area = 288 Sq.Ft.
(d) 255 Sq.Ft.
(c) One Opening = 55 Sq.Ft. Total-11 Openings = 605 Sq.Ft.
Intake
Main Culvert
(f) One Lateral = 41 Sq.Ft. Total-11 Laterals = 451 Sq.Ft.
One Outlet = 12.24 Sq.Ft. (g) 5 Outlets of One Lateral = 61.2 Sq.Ft. Total Area of All Outlets = 673 Sq.Ft.
SIDE WALL
(a) One Passage = 144 Sq.Ft. Total Intake = 432 Sq.Ft.
One Passage = 120 Sq.Ft. Total Intake = 360 Sq.Ft.
One Outlet = 12.24 Sq.Ft. 5 Outlets of One Lateral = 61.2 Sq.Ft. Total Area of All Outlets = 612 Sq.Ft.
One Lateral = 41 Sq.Ft. Total-10 Laterals = 410 Sq.Ft.
Intake
Main Culvert
One Opening = 33.2 Sq.Ft. Total-10 Openings = 332 Sq.Ft.
333 Sq.Ft.
One Side = 144 Sq.Ft. Total Area = 288 Sq.Ft.
255 Sq.Ft.
One Side = 144 Sq.Ft. Total Area = 288 Sq.Ft.
255 Sq.Ft.
CENTER WALL

Fig. 3. Diagram of Filling System.

occasion to observe the effects of the flow; and certain results have been noted which are little short of remarkable.

Both Valves.

Let us examine first the trial, the results of which are shown graphically in Fig. 4. In this trial only the side culvert was used; both halves of the filling valves were opened simultaneously, and

took one minute for the full stroke of 18 feet; the upper pool was at reference +84.8, and the water in the lock at reference +50.9. The time was taken from the first movement of the valve until the water reached the level +84.8 of the upper pool.

The observation shows an elapsed time t^1=14.0 *m*. Of this, one minute is occupied in raising the valves from an opening of 0 to 288 square feet. The curve shows that during this minute the water in the lock rose from 50.9 to 51.7. To determine the true value of *c*, we should consider the flow only during the time the entire culvert area is available, or after the level 51.7 has been reached. This gives $t=t^1-1$ *m*=13.0 *m*.

We have, therefore,

$$H=84.8-51.7=33.1$$
$$h=84.8-84.8=0.0$$
$$t=(14.0-1)\times 60=13.0\times 60.$$

The value of the coefficient *c* from the formula, p. 1, is—

$$c=\frac{2A\ (\sqrt{H}-\sqrt{h})}{t\times a\times\sqrt{2g}}$$

or, for the case in hand,

$$c=\frac{2\times 123000\times(\sqrt{33.1})}{13.0\times 60\times 255\sqrt{64.4}}$$

whence *c*=0.886. The coefficient is therefore much more favorable than was anticipated.

It would appear from the formula that *c* should be constant for any point of the curve, or throughout the entire operation of filling the lock. However, this is not exactly true. Fig. 4 shows that, in the first part of the operation, the lock fills from 51.7 to 70 in 4.6 minutes. We have, therefore, H=33.1, *h*=14.8, *t*=4.6, whence c^1 =0.830. In the latter part of the operation the lock fills from 70 to 84.0 in 6.9 minutes. We have, therefore, H=14.8, *h*=0.8, *t*=6.9, whence c^{II}=0.858. For the remaining part of the curve, from 84.0 to 84.8, we have, H=0.8, *t*=1.5, and c^{III}=1.196. It appears, therefore, that the coefficient increases in value toward the end of the filling, after the maximum velocity in the culvert has been reached. This is apparently due to the inertia of the flowing water, which causes the established velocity to change less rapidly than the head which produces it, and to continue even after the head becomes negative. Toward the top of the curve, when the head is very small, the coefficient *c* is greater than unity, and the velocity therefore greater than that which the head can produce in theory.

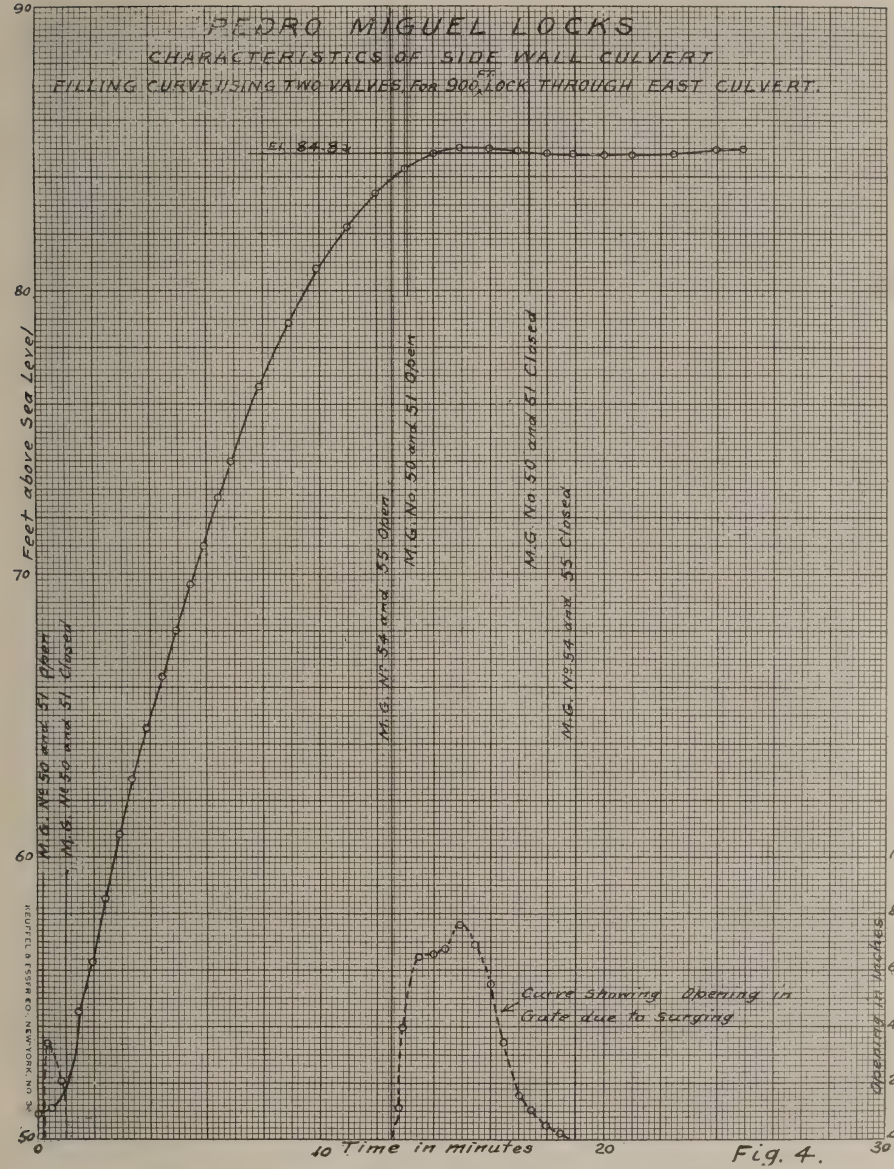


Fig. 4.

Filling Pedro Miguel Lock, using side culvert only, both valves.

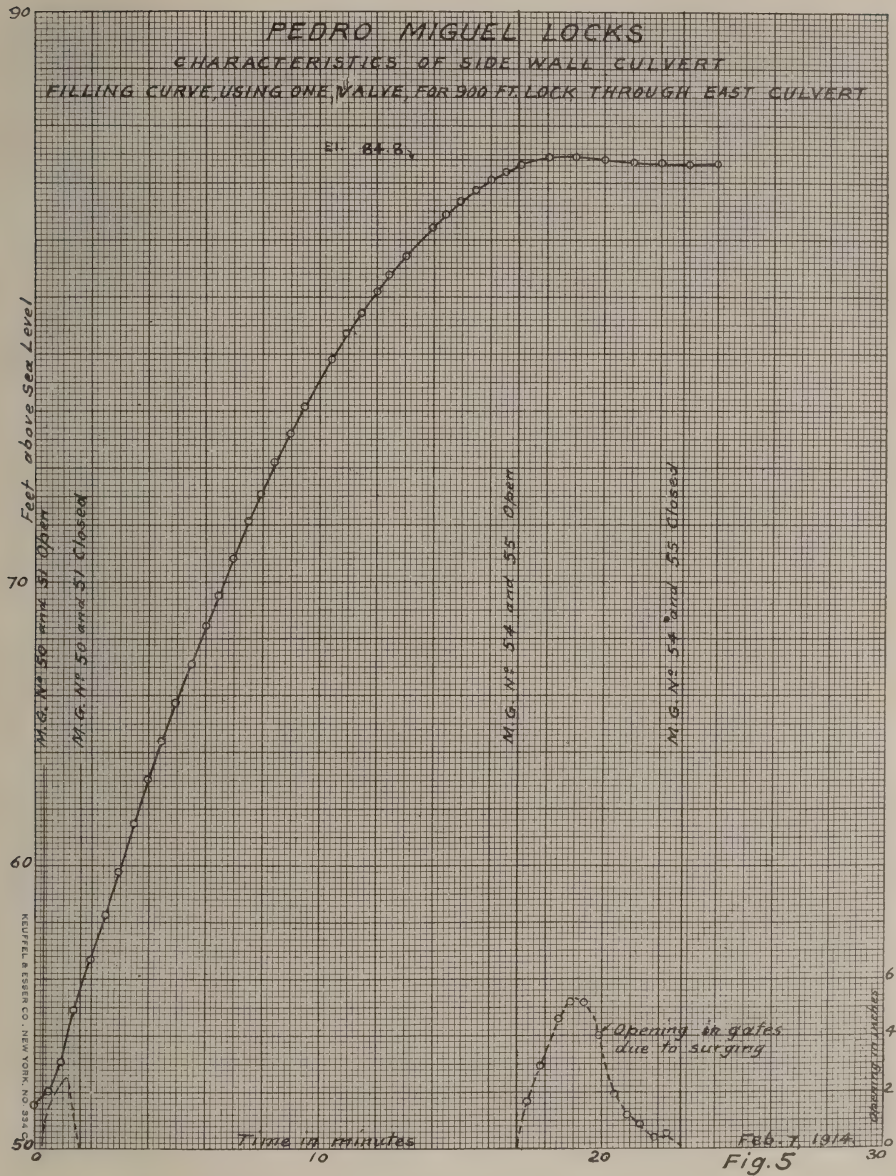
The average value, $c=0.886$, is the important one. If the entire time, fourteen minutes, from the first movement of the valve to the instant of equalization at 84.8 be taken, we shall find $c=0.833$.

It will be noted from Fig. 4 that the upper gates Nos. 54 and 55 open slightly before the upper guard gates, Nos. 50 and 51, and when the water in the lock reaches the level 84.0. The space between these two gates is in communication with the upper pool, and is shut off from the lock chamber. The water in the space, however, does not remain at exactly the same level as the water in the forebay. When the inlet valves are first opened, there is a drop of the water level in the forebay, observed as 0.8 foot, to which the water between the gates does not respond instantaneously; and for a short time there is back pressure against the upper guard gates, as indicated by the curve near the origin of coordinates in Figs. 4 and 5. This soon disappears and there is a positive head against both gates until near the end of the filling, when the water in the chamber rises faster than that between the gates, giving a reverse head on the upper operating gates, Nos. 54 and 55, and causing them to crack at the mitre before the upper guard gates Nos. 50 and 51. In operation it is usual to open the gates as soon as they crack from back pressure, thereby saving the time taken in filling through the height corresponding to the drop in the forebay. In the case shown in Fig. 4, the gates cracked at 84.0; the drop was therefore 0.8 foot, and the time saved would have been 1.5 minutes.

The figure shows also that, after the spontaneous opening of leaves Nos. 54 and 55, 50 and 51, the water in the lock continued to rise until it reached a level 85.0, or two-tenths of a foot above the original lake level, and 1 foot above the level at which the upper gates opened under reverse pressure. This effect is apparently due to over-travel of the water in motion in the culvert, rather than to a surge of the water in the basin and cut above the lock. The latter surge or wave occurs later, as illustrated in Fig. 11.

One Valve Only.

In Fig. 5 is shown the curve for the rise of water when the lock was filled through the side culvert, using only one valve, instead of two. In this case the most contracted area of the waterway is at the valve instead of in the main culvert; and is therefore 144 square feet, instead of 255 square feet. The natural assumption is



Filling Pedro Miguel Lock, using side culvert only, one valve.

that the time of filling will be increased in length, over that observed in the first experiment, in the ratio of $\frac{255}{144}$, assuming the head to be the same in the two cases. The observed time, however, is much more favorable. At the time of trial the upper pool was at 84.8, and the water in the lock at 51.5. The observed interval t^1, from the first movement of the valve until the levels equalized at 84.8, was 18 minutes. As before, it took one minute to raise the valve from an opening of zero to one of 144 square feet. This time should be omitted from the calculation, as well as the corresponding rise of water in the lock, from 51.5 to 53. We have, therefore,

$$H=84.8-53.0=31.8$$
$$h=84.8-84.8=0.0$$
$$t=17.0 \text{ m.}$$
$$c=\frac{2\times 123{,}000\times(\sqrt{31.8})}{17\times 60\times 144\times\sqrt{2g}}=1.177.$$

That is, the coefficient of flow, applied to the most restricted section of the culvert, is greater than unity. At this section, therefore, the average velocity of the water during the process of filling is greater than that due to the head. When the entire time of eighteen minutes is used, and the loss of area until the valve is fully opened is neglected, the value of c is found to be 1.086, or still greater than unity.

As in the case where two valves are used, the coefficient c is slightly more favorable near the end of the operation than near the beginning. Thus, while the water in the lock is rising from +70 to +84.8, we have,

$$H=14.8,\ h=0.0,\ t=11.2, \text{ and } c=1.228.$$

For the lower part of the curve, while the water is rising from 53.0 to 70, we have,

$$H=31.8,\ h=14.8,\ t^1=6.8,\ t=5.8,\ c=1.079.$$

As before, however, the average value is the important one.

This result was so unexpected that observations in verification were taken at the upper lock in the Gatun flight. It was found that with the lake level at 84.9 and the lock level at 55.2, it took sixteen minutes to equalize levels. During the first minute, while the valve was rising and the culvert area increasing from 0 to 144 square feet, the lock rose from 55.2 to 56, or 0.8 foot. We have, therefore,

$H=84.9-0.8=84.1$
$t=16-1=15$ min.
Whence, $c=1.272$.

This value is greater than that found at Pedro Miguel, doubtless because the drop in the forebay, which occurs when the valve is opened, is hardly noticeable at Gatun. This drop is marked at Pedro Miguel, and constitutes a loss of head which subsists nearly through the operation of filling.

EMPTYING THE LOCK.

The Pedro Miguel lock empties through the lower end of the culverts, which are kept at the same level from near the upper end of the lock chamber to the discharge.

Both Valves.

Fig. 6 illustrates the flow when emptying the lock through the side culvert only, with both valves raised. The lock fell from 84.8 to 50.8 in fifteen minutes. Correcting for the time consumed in opening the valves, viz, one minute, during which time the level in the lock fell from 84.8 to 81.7, we have—

$H=81.7-50.8=30.9$
$h=0$
$t=14$ minutes, whence,
$c=0.795$.

If the entire head $H=84.8-50.8=34$, and the entire time $t^1=15$ minutes be used, neglecting the loss during opening of the valves, we shall find $c=0.779$.

One Valve.

The flow is slower when but one valve in the culvert is opened, reducing the most restricted area from 255 to 144 square feet. As in the case of filling the lock, however, the time is not increased in the inverse ratio of the areas, as the formula would indicate. It takes twenty-one minutes to empty the lock from Gatun Lake level of 84.8 to Miraflores Lake level of 51.2. During one minute of this time the valve is rising, the area of flow increasing from 0 to 144, and the level in the lock falling from 84.8 to 83.0. We have, therefore,

$H=83.0-51.2=31.8$
$h=0$
$t=20$ minutes, whence,
$c=1.000$.

If the entire time and head be taken, we have—

$H=33.6$
$h=0.00$
$t=21$ minutes, whence,
$c=0.979$.

The conditions of flow in emptying the Pedro Miguel lock are evidently somewhat less favorable than in filling. This arises perhaps from the fact that the water has to pile up below the discharge in order to assume the surface slope necessary to take it away; and, consequently, the difference in head between the two pools during the operation is somewhat less than the measured difference before the operation and afterward, when the water has come to rest. The small surface of Lake Miraflores is actually raised about 0.09 foot by the lockage prism.

EMPTYING LOWER LOCKS OF FLIGHT.

The discharge openings of the lowest locks at the Gatun and Miraflores flights are given a form different from that adopted at Pedro Miguel, for reasons connected with the different densities of the water on the two sides of the lower gates. The axis of the culvert is turned upward in a vertical plane and the mouth of the culvert is therefore an opening in a horizontal plane, the water discharging upward when emptying the lock. (See Fig. 12.) It is found that the conditions of flow are somewhat less favorable for emptying these locks than for Pedro Miguel.

Thus, for Miraflores, with the water in the lock at +21.00, and tide level below at —1.25, the water fell to —0.75, when the lower gates cracked, in 14.4 minutes, both valves in the side culvert being used. In the first minute of this time, when the valves were rising and the full culvert area becoming available, the water fell from 21.00 to 19.00. We have, therefore,

$H=19.75$
$h=0.00$
$t=13.4$ m., whence,
$c=0.655$.

The coefficient is less favorable than at Pedro Miguel, either because of the shape of the outlet or because of the discharge into a heavier medium. The fact that the water on the two sides of the gates is of different density makes the calculation of c somewhat unreliable, as there was a remaining difference in level of 0.5 foot when the gates cracked, forced open by the heavier water before the

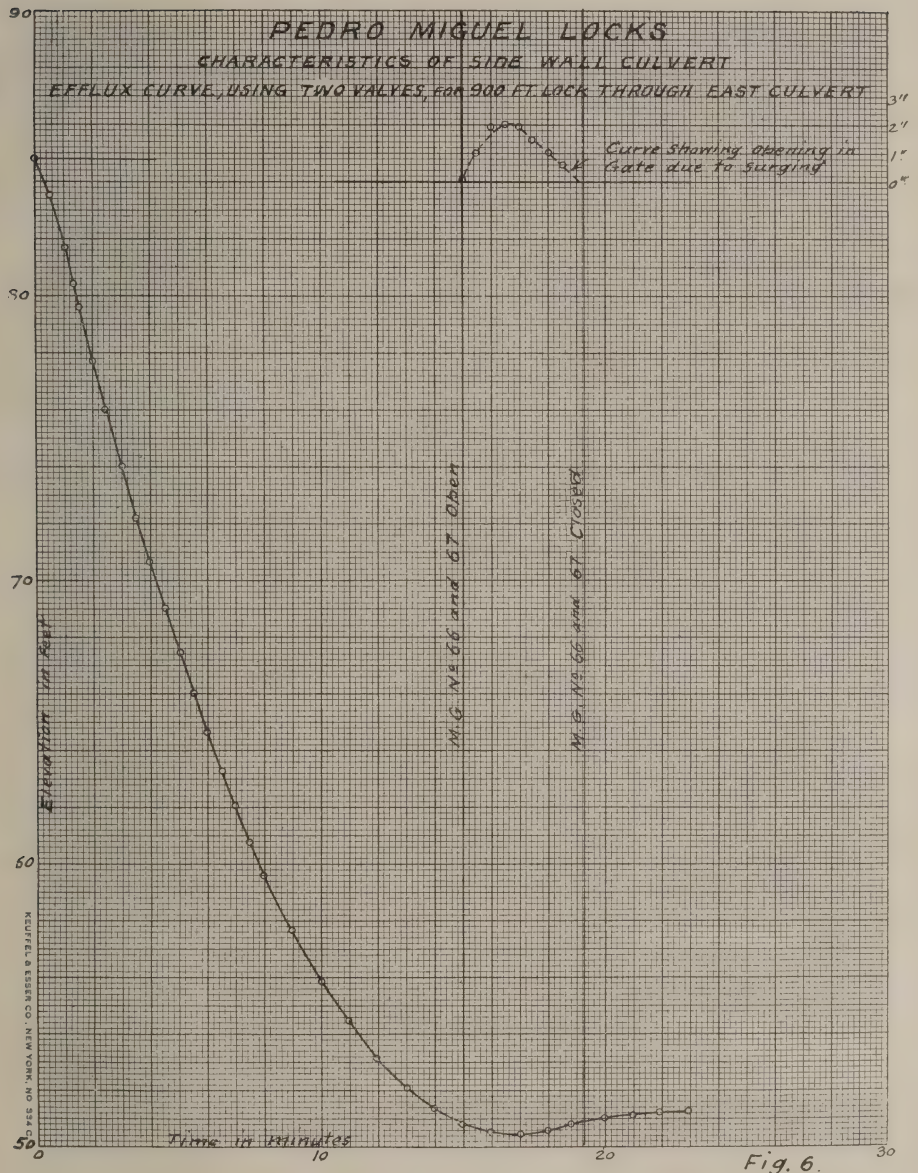


Fig. 6.

Emptying Pedro Miguel Lock, using side culvert only.

levels equalized. The fall H is taken from the original level when the valves were fully opened, to the level when the gates cracked, but no allowance is made for the residual difference in level of $h=0.5$, as in theory the flow through the culvert should cease while a certain head still remains against the upper face of the gate.

EQUALIZING BETWEEN CONSECUTIVE LOCKS.

Miraflores.

Fig. 7 shows the rise and fall of the water in the two locks at Miraflores, when equalized from original levels of +41.8 in the upper lock and +2.5 in the lower lock. The area of the upper lock is 123,000 square feet, and of the lower, 120,000 square feet. The time consumed from the time the valves began to rise until the dividing gates cracked was 9.5 minutes. The side wall culvert was used, both valves being raised. During the first minute, while the valves were rising, the upper level dropped 0.8 foot, and the lower rose an approximately equal amount. The head H reduces therefore from 39.3 to 37.7, and the time from 9.5 to 8.5, if the period of flow through the entire culvert area be considered. We have, therefore,

$$c=2\times\frac{123{,}000\times120{,}000}{123{,}000\times120{,}000}\times\frac{\sqrt{37.7}}{8.5\times255\times\sqrt{64.4}}=0.718.$$

If the entire time, 9.5, and difference in level 39.3 be considered, neglecting the effect of the loss of area while the valves are opening, we have $c=0.656$.

The coefficient of flow in the side culvert, when equalizing consecutive locks of the Gatun flight, is practically the same as determined above for Miraflores.

USE OF MIDDLE WALL CULVERT.

The coefficient of flow through the middle wall culvert is not so favorable as it is in the case of the side wall culvert. Doubtless, the cylindrical valves choke the flow to some extent. Experiments at the upper Gatun Lock are recorded in Fig. 8. In these the valves were raised in 1½ minutes. It is necessary to correct the observed values of t^1 by 1½ minutes, and that of H^1 by the rise of water in the lock during this time. Taking the observations separately, we shall then have—

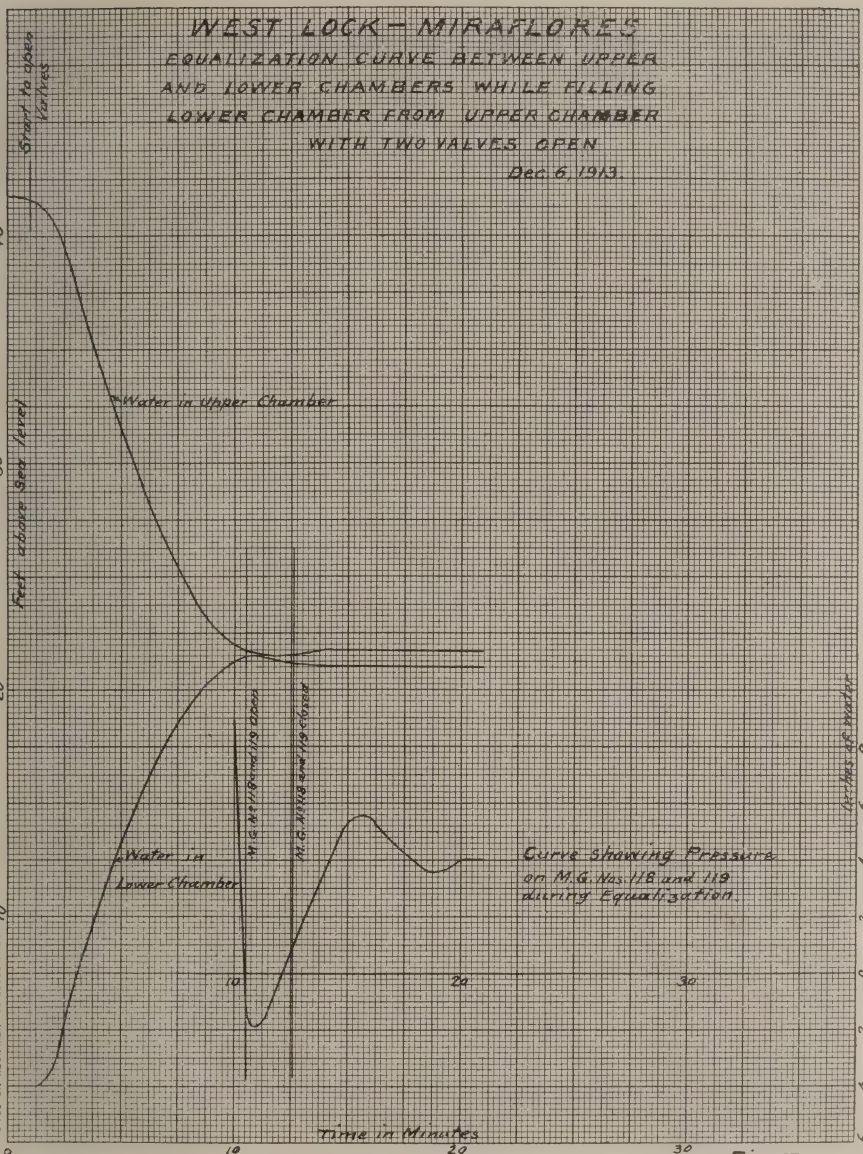


Fig. 7.

Equalizing Miraflores Locks, using side culvert only, both valves.

	t^1	H^1	t	H	$\sqrt{H}$
First	17.83	25.5	16.33	23.5	4.85
Second	19.50	30.0	18.0	27.5	5.24
Third	20.00	31.5	18.5	29.0	5.39
And, Means			17.61		5.16

There being no residual difference in head, $h=0$, and

$$c=\frac{246{,}000\times 5.16}{60\times 17.61\times 255\times\sqrt{2g}}=0.587.$$

USE OF BOTH CULVERTS.

The observation recorded on the same plate for filling the lock using both culverts happens to be very favorable, and, if analyzed, will give a value of $c=0.810$, for the part of the curve after the first 1½ minutes. A mean of six observations taken at different times gives a slightly less favorable value of $c=0.791$.

For the remaining locks of the Gatun flight, we have, as the mean of several observations taken when both culverts were used—

Upper to middle lock, eight observations,

$$\sqrt{H}=7.61 \qquad t=6.20 \qquad c=0.610.$$

Middle to lower lock, eight observations,

$$\sqrt{H}=7.14 \qquad t=6.26 \qquad c=0.562.$$

Lower lock to sea, seven observations,

$$\sqrt{H}=5.18 \qquad t=7.45 \qquad c=0.685.$$

It will be observed that the flow is less favorable from the middle to the lower lock than from the upper to the middle, although the arrangement of the culverts is practically identical. The water in the lower lock is salt when equalization takes place, that in the middle lock is much less salt, and may be nearly fresh, according to the direction of the lockages immediately preceding the observation. Perhaps the density of the medium into which the culverts discharge has an influence on the freedom of flow. At all events, the observations indicate that it takes more time to equalize relatively fresh water into salt water than into brackish or nearly fresh water, other known conditions being the same.

The flow from the lower lock to the sea is also somewhat less favorable than from the lake into the upper lock, as is also found

at Miraflores. In both cases the shape of the side wall culvert at the outlet may have an influence on the rapidity of flow.

It is very probable that a long series of observations would show somewhat different values of the coefficients as the true ones; the oscillation of the water level in the chamber which is filled sometimes makes the gates crack earlier than they would were there no such surging, and the presence of large ships in the lock may and probably does impede the flow to a certain extent. The values of c given below are those which the writer's observations and analyses indicate as sufficiently correct for the 900-foot lock chamber.

For	Value of c
Filling upper lock of a flight and single lock—	
Side wall only (2 valves)	0.86
Both walls (4 valves)	0.78
Equalizing between locks—	
Side wall only (2 valves)	0.70
Both walls (4 valves)	0.60
Emptying lower lock of flight—	
Side wall only (2 valves)	0.66
Both walls (4 valves)	0.69
Emptying Pedro Miguel lock—	
Side wall only (2 valves)	0.80
Both walls (4 valves)	0.79

The above values are an approximate mean of the values determined at the different locks under similar conditions. The observations indicate a slight difference in the coefficients of flow for equalization from the upper to the middle, and from the middle to the lower lock at Gatun; and from the upper to the lower lock at Miraflores, the flow being somewhat better in the first case than in the others. Speaking roundly, it may be said that, with the usual difference in levels, it takes, using both culverts, 7.5 to 8.0 minutes to fill the upper lock of a flight, or the Pedro Miguel lock, 6.5 to 7.0 minutes to equalize between locks, and 7.5 to 8.0 minutes to empty the lower lock of a flight, or the Pedro Miguel lock. When only the side culvert is used with both valves, these intervals become about 13.5 m., 12.5 m., 17.5 m., and 15 m., respectively. In all cases the time is counted from the first movement of the valve, and includes therefore the time consumed in opening the latter. Evi-

dently the locks can be filled or emptied in a satisfactorily short time, even when only one culvert is used.

FLOW OF WATER INTO CHAMBER.

When any one of the locks is filled, using the side wall culvert only, the discharge is noticeably greatest at the opening in the floor farthest from the culvert, and nearest the middle wall. The velocity of flow in the laterals evidently carries the water past the first two or three openings without giving them their full share of the discharge. Were the laterals lengthened, doubtless this effect would become less and less marked; and, finally, the maximum discharge would be found at some opening nearer the source of supply. With the proportions which they have, however, there is no doubt that the openings nearest the middle wall discharge a larger volume of water than those nearer the side wall, when the side culvert only is used. The result is a slight slope of the water's surface, which draws vessels away from the middle wall toward the side wall. (See Figs. 9 and 14.) This effect is not serious and disappears when both culverts are used. (See Fig. 10.)

The side culvert occupies much the same relation to the laterals which draw from it as does each lateral to the openings through which it discharges. The side culvert is, however, made longer, and it is not plain to the eye which one of the laterals gets the largest share of the water at any instant. The ones farthest upstream discharge first, the boils from these beginning to show in about one minute after the first motion of the valves. The flow becomes apparent to the eye progressively down through the length of the lock, the lateral at the lower end—that is, the one farthest from the filling valve, showing flow last of all. At some instants during the filling, one of the laterals may seem to give a more violent discharge than the others; but it will not always be the same lateral. It may be observed during the same operation that the greatest flow appears to come at one instant from the lateral nearest the valve, at another instant from the one farthest from the valve, and at another instant from some intermediate lateral. The writer does not feel that the observed facts justify a statement that the discharge from the main culvert into the laterals is uniformly greatest at any point of the length of the culvert. There is no tendency to draw a vessel in either direction along the axis of the lock.

The process of raising the water level in the lock is accompanied

by some little agitation of the water surface, especially when one culvert only is used. (See Figs. 9 and 10.) This is not sufficient to disturb boats, except as indicated on page 26, and when both culverts are used even the smallest boat is reasonably quiet and perfectly safe. In lowering the water level there is no disturbance apparent at all, the surface being perfectly placid and without visible current, whether one or both culverts are used.

BACK PRESSURE ON THE SAFETY GATES.

It will be noted (Fig. 1) that the space between the two sets of lower gates, called the "safety gates" and the "lower gates," is

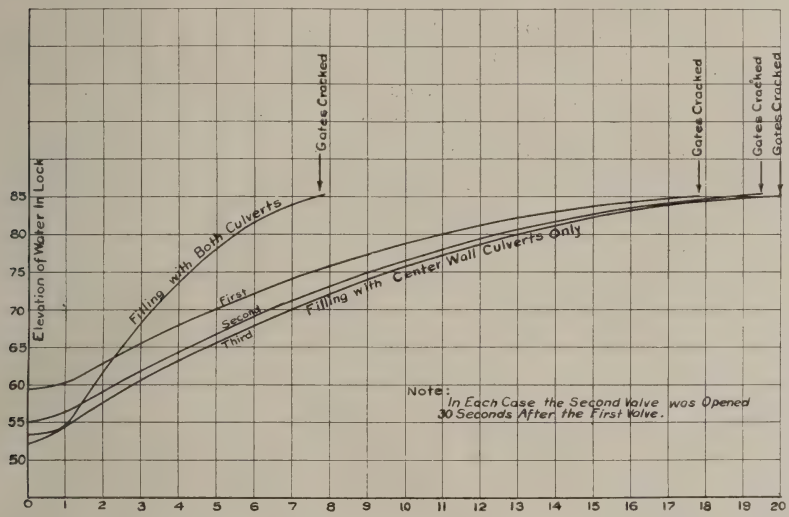


Fig. 8. Filling of Upper West Lock, Gatun, using middle culvert only, and using both side and middle culverts.

regulated by a special culvert, called the T-culvert, which is usually open to the main side wall culvert above the lower valve, but which may be closed by a valve of its own, to shut off the water pressure in the lock from the lower gates. Such a separation would be necessary when, for any cause, the lower gates might be out of commission and it might be desired to operate the lock, using the safety gates only as a lower barrier. When the lock was first filled, the valve was raised in its well and the T-culvert, which has a cross-section of 65 square feet, was unobstructed.

In fixing this cross-section, an attempt was made to reach such a relation to the sectional area of the side wall culvert that the

space between the two sets of gates would fill and empty at about the same rate as the main lock chamber. When the lock was first filled it was found that, with the valve fully raised, the space between the gates filled faster than the chamber. The quicker filling of the space between the leaves produces back-pressure on the safety gates, and may give rise to undesirable stresses in the gate-operating mechanism. The difficulty has been overcome (a) by opening the safety gates as soon as the vessel has been made fast in the locks and the upper gates closed, or (b) by choking the opening of the T-culvert with the valve. By trial, a position of the valve has been found which causes the space between the gates to fill more slowly than the lock chamber, and to empty more rapidly, thus keeping a positive pressure on the safety gates at all times. When both side and middle culverts are used, this position of the check valve in the T-culvert is different from that occupied when one culvert only is used. The safety gates are habitually opened immediately after the valves, when the water is to be lowered.

BACK PRESSURE ON UPPER GATES.

Note has already been made (page 8, ante,) of the fact that the upper gates are subject to reverse pressure on account of the over-travel of the water in the culverts, which causes the level of the water in the lock to rise somewhat higher than that in the forebay. At Pedro Miguel, when the water is drawn from the relatively narrow entrance basin and canal, the reverse head may reach about 0.8 foot, the water rising in the lock to a level 0.2 foot higher than the original level of the water in the forebay, while the latter is still depressed below its original level. This phenomenon is due to the local over-travel of the water in motion in the filling system, and is not to be confounded with the wave set up in the canal prism when the filling valves are opened. At Gatun and Miraflores a similar over-travel of the water, sufficient to cause the upper gates to open from back pressure, has been noticed; the water in the forebay at these locks does not drop so far as at Pedro Miguel while the lock is filling, and the reverse head on the upper gates is therefore much less marked.

CHANGE OF LEVEL IN CANAL.

The drop in the forebay at Pedro Miguel when the filling valves are opened sets up a slope of the surface in the canal prism, which

causes a general flow toward the lock. When both culverts are used this flow ordinarily reaches 13,000 f. s. If this draft were to continue sufficiently long to establish a steady flow in the canal prism, there would be a current of about 1.0 f. s. in the narrow parts of the Culebra Cut. Above the lock for 3,400 feet, the canal was widened to form a basin from which the filling water is drawn; and the loss of water from this basin is replaced by flow through the canal prism; but a much longer time is occupied in replacing than in withdrawing it. The current in the Cut is therefore not as great as 1.0 f. s., but is nevertheless noticeable, especially near lock.

A large volume of water is set in motion toward the lock, only

Fig. 9. Filling Pedro Miguel Lock through side wall culvert.

to find its motion arrested when the filling is completed. A rise of water results in the forebay and lock, which reaches its maximum about one hour after the filling begins, or about forty-seven minutes after it has been completed when one culvert only is used. When two culverts are used, these intervals are reduced about fifteen minutes. This rise corresponds in amount quite closely to the drop which occurs after the filling valves are opened. The total vertical oscillation is about 1.0 foot when only when one culvert is used, and 1.25 feet when two are used.

Fig. 11 shows an hydrograph taken in the forebay, there having been one lockage with one culvert and one with both cul-

verts. It will be seen that the recurring effect of the wave is felt for several hours.

Lockages at Pedro Miguel are indicated on the hydrographs taken at the observing station at Juan Mina, on the Chagres, nearly 5 miles above Gamboa and 13 miles from the lock.

EFFECT OF DIFFERENCE IN DENSITY OF WATER.

When the lowest lock of the Gatun or Miraflores flight has been filled, the water in the chamber is a mixture of fresh and salt water, while the pool below has full salinity. The culverts through which equalization of levels takes place are at the base of the walls, 40 feet below the low water surface. Flow through the culverts should cease, in theory, when the weight of the column of water, measured above the axis of the culvert, is the same on the lock side of the culvert opening as it is on the sea side, since then the unit pressure at the culvert level is the same in both directions. As the water in the lock is lighter than that in the sea, the column on the lock side of the opening, and therefore above the lower gates, is somewhat higher than that on the sea side, or below the gates. Calling the density and the height above the axis of the culvert

δ and h, above the gates, and
δ^1 and h^1, below the gates,

the unit pressures at the culvert axis will be equal when $\delta h = \delta^1 h^1$, the height entering in the first power.

The pressures on the two sides of the gate, above the culvert axis, are measured by $\frac{\delta h^2}{2}$ and $\frac{\delta^1 h^{1\,2}}{2}$, the height entering in the second power. It appears therefore that the unit pressures in the culvert may equalize, and the flow cease, while there is still a difference in pressure on the two sides of the gate leaf. This residual difference in pressure may be sufficient to prevent moving the leaf. On pages 85 to 99 of the Annual Report of the Isthmian Canal Commission for 1911 will be found an extended analysis of the effect of this possible difference in pressure, and of the means of neutralizing it. For present purposes, it is enough to say that the lower ends of the culverts at lower Gatun and Miraflores were turned upward and the discharge openings placed in a horizontal plane at the elevations at which the theory indicated that the remaining pressures on the gates would be within manageable limits. (See Fig. 12.)

Fig. 10. Filling Pedro Miguel Lock through side and middle wall culverts.

This design for the discharge may have been an unnecessary precaution. No existing examples could be found where the difference in density of the waters on the two sides of a lock gate had caused any difficulty in opening the gate; even in the Manchester Canal no trouble was in evidence. On the other hand, the conditions there did not approximate in severity to those in Panama; and the precaution cost so little that it was considered wise to take no chances. In point of fact, no trouble is now experienced in maneuvering the gates, due to residual head. When the lock is emptied the water in the lock usually ceases to fall while it is still about 6 inches above the sea level outside. The water in the culvert continues to flow after the unit pressures have equalized, due to the acquired kinetic energy; and this overtravel continues until the pressure on the downstream side of the gates is greater than on the upstream side, and the leaves crack from back pressure. As stated above, however, this generally occurs before the actual equalization of levels, which does not take place until the gates are opened. Whether the over-travel would have been sufficient to equalize pressures on both sides of the leaf, had the culvert discharge been left at a low level, as at Pedro Miguel, is a matter of speculation. The important thing is that the trouble which was feared is not felt; perhaps because of the design and perhaps because of the over-travel of the water, which was not anticipated when the design was made.

CURRENTS IN THE LOCKS.

The difference of density of the water in the locks produces some curious phenomena. When the lower gates of the sea locks are opened after equalizing pressures, the relatively fresh water in the lock chamber rushes out to sea on top of the salt water in the channel below, giving rise to a surface current which may reach 2 feet per second and last for forty-five minutes. This surface current is accompanied by a counter-current in the lower levels, produced by the salt water rushing into the lock. The effect of the combination is not troublesome to a large steamer, but is felt by light draught vessels, such as barges, tugs, etc., since the surface current spreads out immediately after passing the splay of the lower wing wall, and tends to throw the boat out from the middle wall and toward the bank. The effect is not serious, however. A curve showing the observed currents is shown in Fig. 13.

A similar effect is felt when the gates separating two locks in

the flight are opened after equalizing levels. There is a noticeable surface current from the upper lock, which contains the lighter water, to the lower lock, which contains the denser. Eventually the waters arrange themselves according to the weight, and all is quiet; but the current while it lasts is felt by the towing locomotives when moving a boat from the lower into the upper lock.

It may be deduced from the foregoing that the salt water climbs

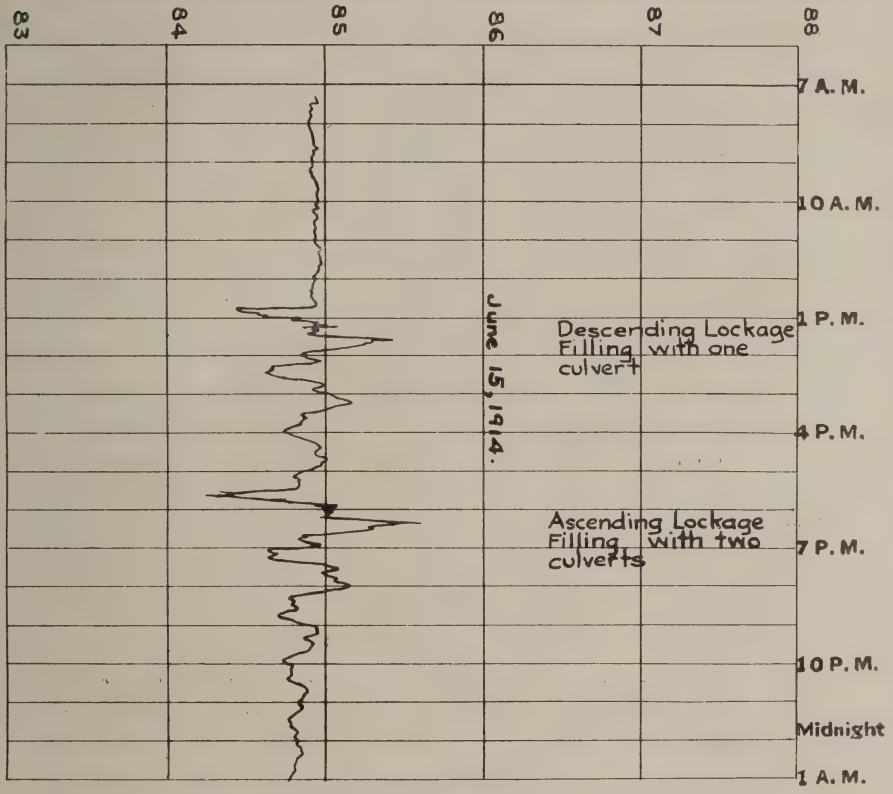


Fig. 11. Oscillation of Water in Forebay, due to Filling Pedro Miguel through one or two culverts.

up the lock flights and appears in the pools above, although the bottoms of these pools are above sea level. As one result of this ascent, Miraflores Lake has become so salt that it can not be depended upon for water supply, and the Canal authorities have been forced to extend the draught-pipe line to Gamboa, and draw from the Chagres River there. Eventually, perhaps, the salinity will become noticeable in Gatun Lake, although the surface will prob-

ably always remain reasonably fresh. In Miraflores Lake now (October 1, 1914), the surface has a chlorination of three hundred parts in one million, while the bottom layers run to one thousand parts or more. Normal sea water has about twenty thousand parts to the million.

The draw of the vessel away from the middle wall, alluded to on page 18, ante, as observed when filling from the side culvert only, is much more noticeable in the lower locks at Gatun and Miraflores than it is elsewhere. In these locks only is it of such force that the towing locomotives can not overcome it, and hold the boat central in the lock. There it is advisable to moor the boat against the side wall before filling, if only the side culvert is to be used. When both the side and middle culverts are used, there is no tendency of the boat to draw to either side of the chamber. Doubtless, the exaggerated effect noted in these locks is due in part to the fact that the lower lock contains practically pure salt water when the relatively fresh water from the upper lock is admitted underneath; and the lifting effect of the upward flow is increased by the lightness of the water admitted at the bottom. Figs. 14 and 15 illustrate the difference in the upper and lower locks at Miraflores. In both cases the vessel was held by the towing locomotives only. They failed to keep the tow central in the lower lock, but had no difficulty in doing so in the upper lock. There is no tendency to move the vessel when the lock is emptying, either through one or both culverts; nor is there a tendency at any time to move the vessel along the axis of the lock.

VELOCITY IN CULVERTS.

The total lockage prism at Pedro Miguel, with upper pool at +86 and lower at +54 is, say, 3,936,000 cubic feet for the 900-foot lock. As this is moved in 7½ minutes through two culverts with total sectional area of 510 square feet, it must have an average velocity of 17.1 feet per second throughout the entire time of filling. When the velocity is greatest, the surface in the lock rises or falls at the rate of about 6.5 feet per minute. At this period of the operation the mean culvert velocity is 26.1 feet per second. The culverts are lined with cast-iron plates at the valve chambers, but not elsewhere.

LEAKAGE FROM GATUN LAKE.

At each end of the summit level there is leakage through the gates and valves of the locks; and at Gatun there is also leakage

through the spillway. As the water supply during the dry months of the year may require careful conservation, it is important to know how much is lost through that leakage. Accordingly, it has been carefully measured. At the spillway a light dam was built in the channel below the main concrete dam through which the leakage occurs, and all the flow in this channel was collected

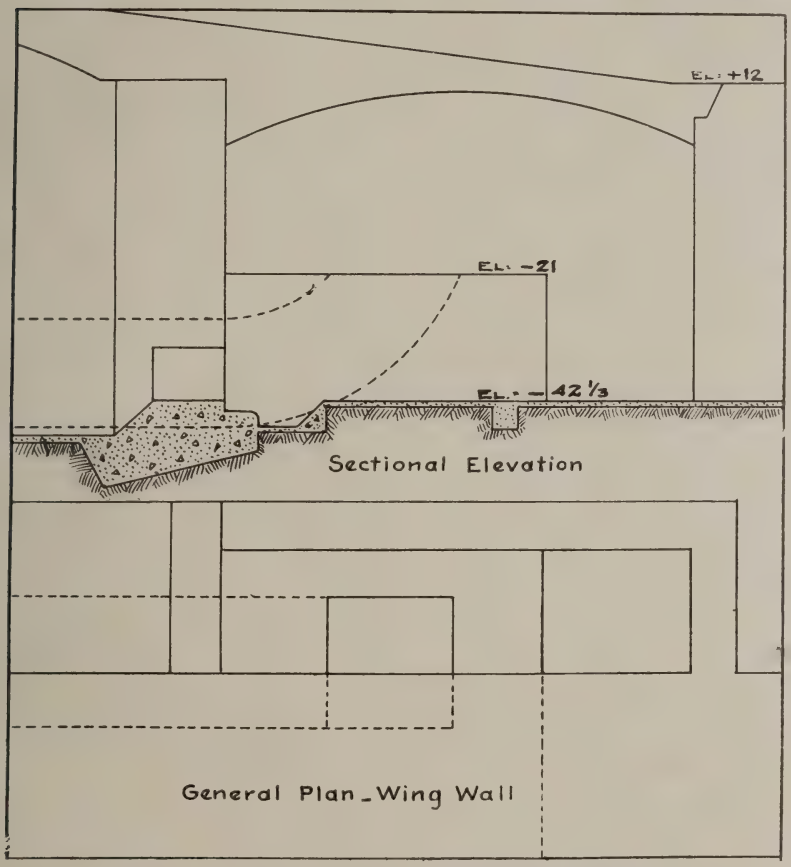


Fig. 12. Lower end of Side Wall Culvert, Gatun.

at a weir where it could be measured. The result showed leakage varying from 1.34 f. s. at 7 a. m., to 7.90 f. s. after the hot part of the day, at 4 p. m., when the steel crest gates have expanded under the action of the sun. The average leakage for forty-nine hours was 3.65 f. s. The lake was at normal level of +85.

At Pedro Miguel and Gatun observations were made on the water

levels in the locks. When the upper and lower gates and valves were kept closed with water at the upper pool level, and the twin locks kept in communication through the cylindrical valves in the middle wall, the surface of the water in the lock chambers fell an average of rather less than 1 foot in twelve hours. The total loss to the lake is the amount indicated by the fall in the surface of the lock chamber, plus the amount which leaked into the

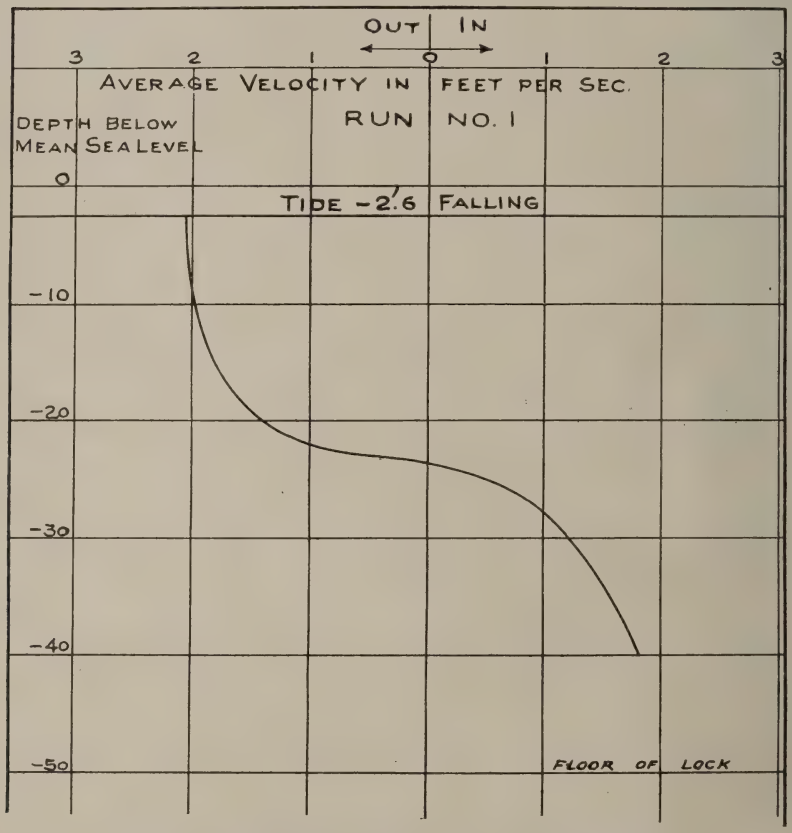


Fig. 13. Curve showing Currents below Miraflores Lock.

chamber through the upper gates and valves under the head against the upper barriers.

As the head against the lower gates and valves was from 57 to 27 feet, while that against the upper gates and valves was small, being from 0 to 1 foot during most of the observations, the correction due to leakage into the chamber was not important. When applied it makes the average leakage through both twin locks at

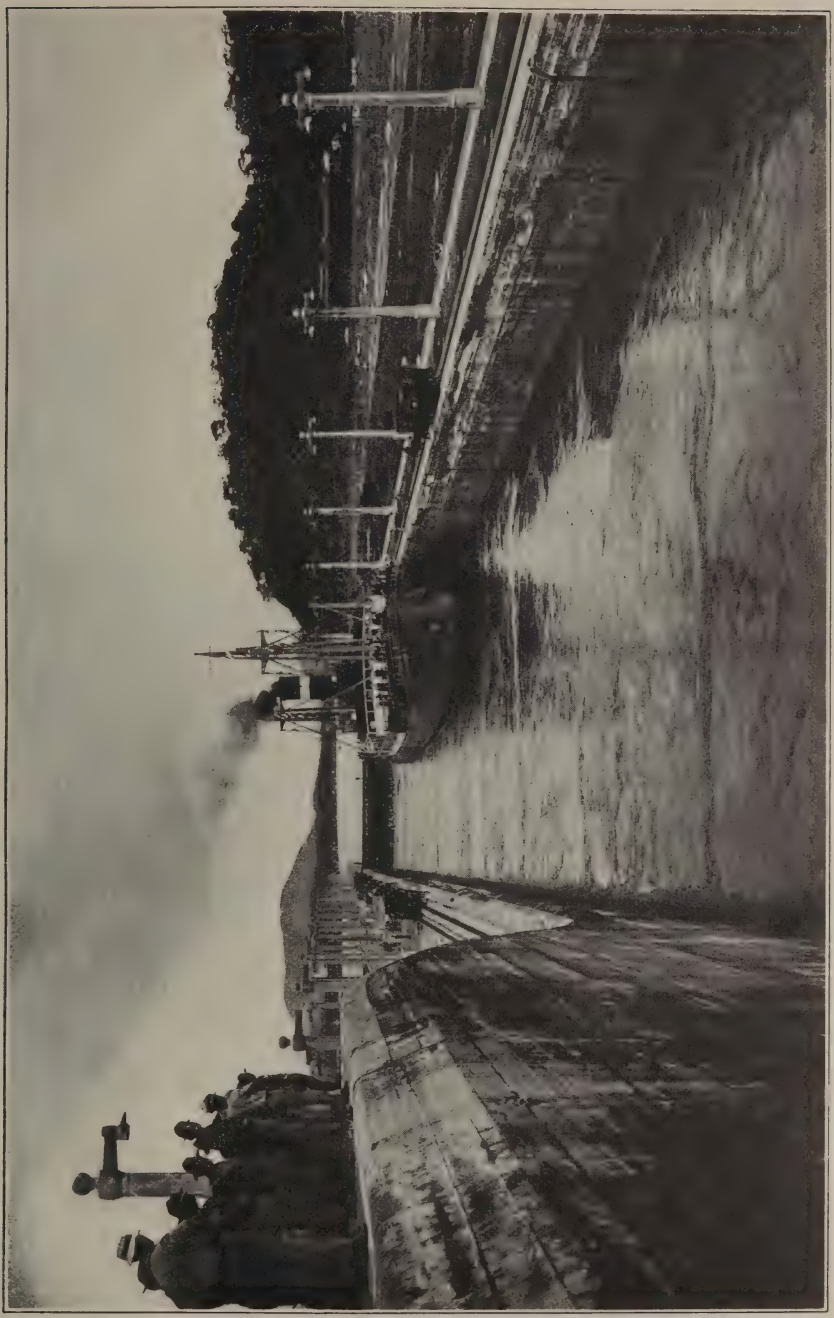


Fig. 14. Steamer *Santa Clara* in lower west chamber, Miraflores.

Gatun 5.93 f. s., and at Pedro Miguel 4.99 f. s. At each locality this is the measure of the leakage through two pairs of gates and three pairs of Stoney valves, and speaks sufficiently well for the tightness of these parts.

The total loss by leakage through the terminal locks and spillway of the summit level is therefore—

	Second feet.
At Pedro Miguel	4.99
At Gatun lock	5.93
At Gatun spillway	3.65
	14.57

An allowance of 275 second-feet was made for this leakage, in estimating the water supply available for all purposes.

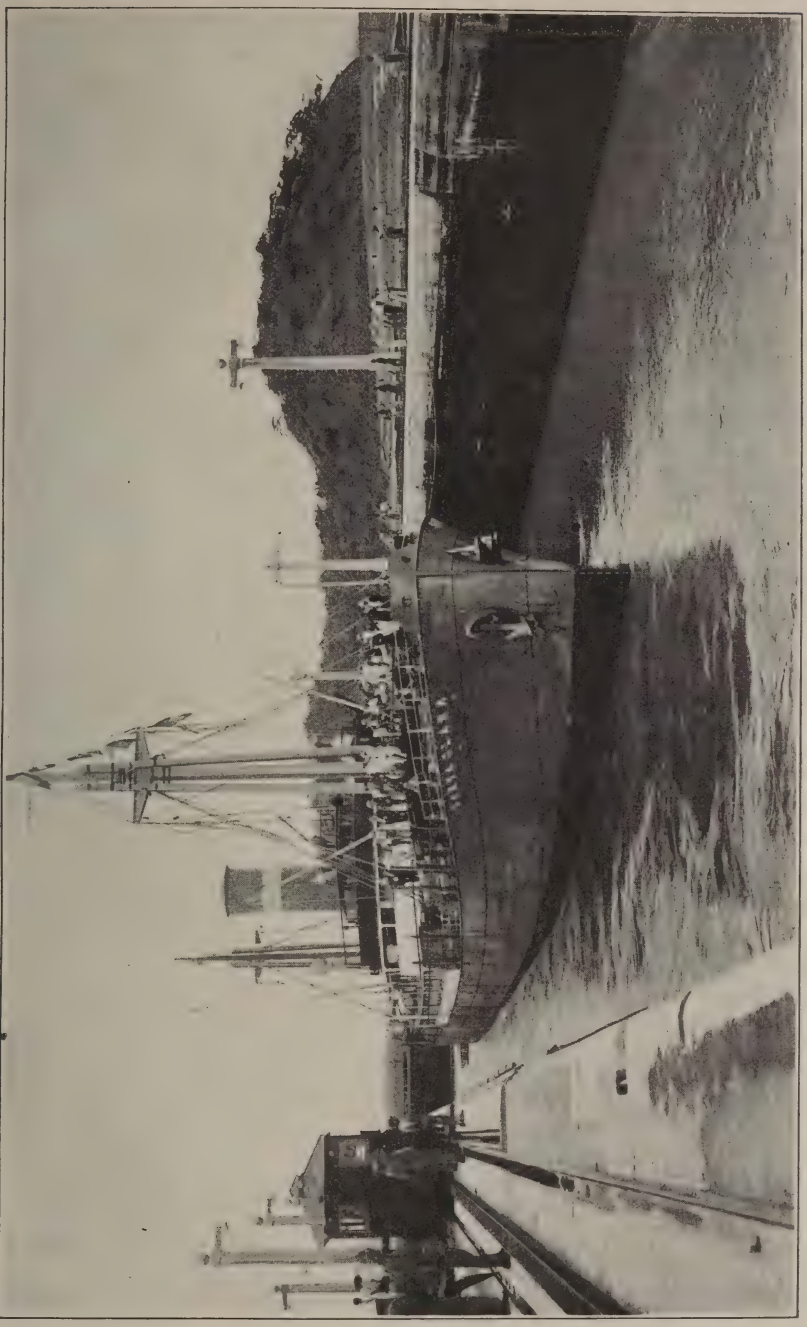


Fig. 15. Steamer *Santa Clara* in upper west chamber, Miraflores.

Percolation and Upward Pressure of Water.

BY

Capt. W. A. MITCHELL,
Corps of Engineers, U. S. Army.

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1. In the design of dams, provision for percolation and upward water pressure is fairly new. Although it has been known for years that some provision was necessary, no accurate allowance has been made for upward pressures. Very little information is obtainable. The engineer has made his dam more than safe in other directions and has trusted to Providence that the percolation would not be too great and the upward water pressure would not be sufficient to more than counteract his precautions.

2. There are four ways that dams may fail:

(*a*) Overturning.
(*b*) Sliding.
(*c*) Crushing.
(*d*) Blow-out.

The pressure of water under the foundation must be considered in the first three, for it is seen that—

(*a*) The danger of overturning is increased because the weight of concrete is decreased by the upward water pressure.

(*b*) The danger of sliding is increased because the weight for friction is decreased.

(*c*) The danger of crushing is increased in the general case because the upward water pressure in the general case transfers the center of pressure nearer the toe of the dam, and this increases the unit pressure at this toe and increases the unit crushing effect.

Percolation must be considered in order to prevent (*d*) blow-out.

3. In order to design intelligently any dam or lock, it is evident that information must be available on the following subjects:

Adhesion.
Percolation.
Upward water pressure.

These are related, but bear no constant relation to each other for different materials; consequently, a separate set of results should be available for each material.

4. In discussing these subjects, I have first stated in each case the conclusion which seems to me logical as shown by experiment or as a result of the information available, and have then given such information or opinions as are available supporting or opposing this conclusion. Opinions are numerous, and do not always support the conclusion. Real facts and experiments are rare: in fact, the Ohio River Board found it necessary to make many experiments in order to supplement the limited information available.

5. The subject is discussed in this paper as follows:

Paragraphs 6 to 12: Adhesion. This is necessary in order to determine what value can be given to adhesion of concrete to concrete or of concrete to rock.

Paragraphs 13 to 53: Upward water pressure. This is necessary in order to decide the amount of upward pressure under foundations and the necessary allowance for this pressure in design of dams.

Paragraphs 54 to 81: Percolation. This is necessary in order to decide as to percolation factor for safety against blow-out; it will also be useful to others in determining amount of seepage and leakage, though this is not of prime importance to the present subject.

Paragraphs 82 to 99: Summary of conclusions.

Paragraphs 100 to 116: Application of these conclusions to design of dams.

ADHESION.

6. *Conclusion: Little or no allowance can safely be made for adhesion of concrete to concrete or of concrete to rock.*

7. Tests were made in 1913 by Professor L. J. Johnson, of Harvard University, showing shearing strengths of joints between old and new concrete. He placed a plug of new concrete inside a ring of old concrete; let it set, and pulled it. The following table shows results:

No. of specimen.	Age of ring, days.	Age of plug, days.	Total load, pounds.	Unit bond shear, pounds per sq. in.
1	35	28	122,000	800
2	14	7	92,400	605
3	21	14	90,200	591

Comment: These are laboratory tests and are not considered applicable in the field.

8. A paper was recently submitted by N. St. G. Robinson before the Institute of Civil Engineers, in London, on the subject of ad-

hesion of old and new concrete. Assuming the solid prism to have an efficiency of 100 per cent, the following results were obtained:

	Per cent
Face wetted only	38.3
Face roughened and wetted	56.2
Face roughened and grouted	85.5
Face treated with acid	82.0

Comment: These are laboratory tests, and do not accord with tests made at Pittsburgh under field conditions.

9. On pages 3019-3021, Report of the Chief of Engineers, Part 4, 1895, are recorded results of many tests made by Colonel Poe of the adhesion of cement mortar. The following are quoted:

	Lbs. per sq. in. mean adhesion.
1 Portland cement to 1 sand—	
Kelley Island limestone, sawn, 28 days	241
1 Portland cement to 2 sand—	
Kelley Island limestone, sawn surfaces, saturated before used, 28 days	169
1 Portland cement to 1 sand—	
Potsdam sandstone, cleavage surface (mortar generally separated from stone, leaving clean surface) 28 days	78
1 Portland cement to 0 sand—	
Cohesive tests of mortar alone, 28 days	686
1 Portland cement to 2 sand—	
Cohesive tests of mortar alone, 28 days	467

Comment: These are very complete tests, though made twenty years ago. They are not real field tests.

10. On January 3, 1914, two tests were made by Junior Engineer Breck under direction of Colonel Shunk, the District Engineer Officer at Pittsburgh, Pa., to determine the adhesion in shear of concrete joints. Two pairs of blocks were molded in place on the top of the land wall of the lock about 3½ months before the test. In one test, the land wall was clean and smooth when the blocks were molded on it. In the second test, the land wall had been somewhat roughened by picking with an ordinary pick. The blocks were pushed apart by the hydraulic jack.

Omitting all question of moment and unusual stresses, and assuming that the resistance was due only to adhesion and to the friction of the two surfaces, which gives the maximum possible value to these two, we find that the required force to rupture the bond amounted to about 50 pounds per square inch in each case.

11. On February 5, 1914, further tests were made at Pittsburgh

to determine adhesive tension between concrete joints. The apparatus consisted of eight hydraulic jacks which lifted blocks of concrete having joints 3 by 3 feet adhering to the land wall at Lock No. 1, Monongahela River. These concrete blocks were molded about 4½ months before the test. The joint was examined after each experiment, and in no case was there any evidence at all of poor bond or weakness in the joint.

After making the test reductions, etc., the following are the final results obtained from adhesion:

Experiment No. 1: Surface of land wall washed with water; ultimate strength of adhesion 41 pounds per square inch.

Experiment No. 2: Surface of land wall washed with acid; ultimate strength of adhesion 2.4 pounds per square inch.

Experiment No. 3: Surface of land wall washed with water, joint cemented with mortar; ultimate strength of adhesion 7 pounds per square inch.

Experiment No. 4: Surface of land wall picked and washed with water; ultimate strength of adhesion 47 pounds per square inch.

Comment: These two sets of experiments were as near to field conditions as could be arranged. The concrete was the usual field mix, was thrown into the forms in the usual way, and no more or no less care was given than is usually given in the field. Laboratory experiments would give very different results, and likewise very different results would have been obtained in these experiments if field conditions had not been actually simulated.

12. The committee appointed to inquire into the cause of failure, in April, 1895, of the Bouzey Dam near Epinal, France, stated:

1. The masonry was exposed to tensions which exceeded its powers of resistance on account of the defective adhesion of the part built in 1880 to that built in 1879.

2. The catastrophe to this dam shows that it is necessary to so design masonry dams that the masonry is not exposed to any tension.

UPWARD WATER PRESSURE.

13. No one seems to have considered thoroughly the question of upward pressure on dams on sand or gravel foundations. Of course, it is generally possible to find rock foundation for a dam; but the Ohio River Board finds it necessary to build dams on sand and gravel foundations, and must consider upward pressure on such foundations.

14. The tendency has generally been to avoid the subject, although some slight consideration has been given to the upward

water pressure under foundations on rock. Wegmann practically ignores it. Others make vague statements to the effect that the foundation must be so constructed that there can be no upward pressure, which is not practicable in practice. However, engineers are now generally aware of the necessity of careful consideration of upward water pressure on rock foundations.

15. *Conclusion: Water pressure is transmitted through concrete, though quite slowly. If allowed free exit, the pressure is practically negligible.*

16. Mr. David M. Francis, past president of the American Society of Civil Engineers, in October, 1888, read before the American Society of Civil Engineers a paper giving results of experiments to find if water pressure could be transmitted downward through Portland cement mortar, mixtures 1 to 2. The same effect would naturally be obtained by lateral pressure, and would result in upward water pressure. He used the apparatus shown in Fig. 1. Great precaution was taken to make this apparatus water-tight. The results of his experiments are given below:

Experiment 1: From April 11, 6 p. m., to April 12, 9.15 a. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 65.5 pounds. During the final one hour and twenty minutes there was no change in the pressure at gauge M.

Experiment 2: From April 12, 9.16 a. m. to April 12, 3.27 p. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 64.0 pounds. During the final one hour and thirty minutes there was no change in the pressure at gauge M.

Experiment 3: From April 12, 3.28 p. m., to April 13, 11.16 a. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 62.6 pounds. During the final one hour there was no change of pressure at gauge M.

Experiment 4: From April 13, 11.22 a. m., to April 13, 5.35 p. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 59.75 pounds. During the final one hour and twenty-six minutes there was no change of pressure at gauge M.

Experiment 5: From April 13, 6.02 p. m., to April 14, 8.01 a. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 57.25 pounds. Period during which pressure at gauge M was stationary not observed.

Experiment 6: From April 14, 11.42 a. m., to April 16, 9.14 a. m. Pipe H closed, pipe I open. Pressure at gauge M went up from 0 to 46.5 pounds. During the final one hour and nine minutes there was no change of pressure at gauge M.

Experiment 7: From April 16, 9.30 a. m., to April 19, 8.12 a. m.

Pipes I and H both closed. Pressure at gauge M went up from 46.5 pounds to 72.5 pounds. Gauge N not then attached. During the final twenty-one hours and forty-five minutes no change of pressure at gauge M.

Experiment 8: From April 19, 8.12 a. m. to April 20, 10.45 a. m. Pipe H closed, pipe I open. Pressure at gauge M went down from 72.5 pounds to 39.75 pounds. During the final two hours and forty-two minutes no change of pressure at gauge M.

Experiment 9: From April 20, 10.45 a. m., to April 23, 11.30

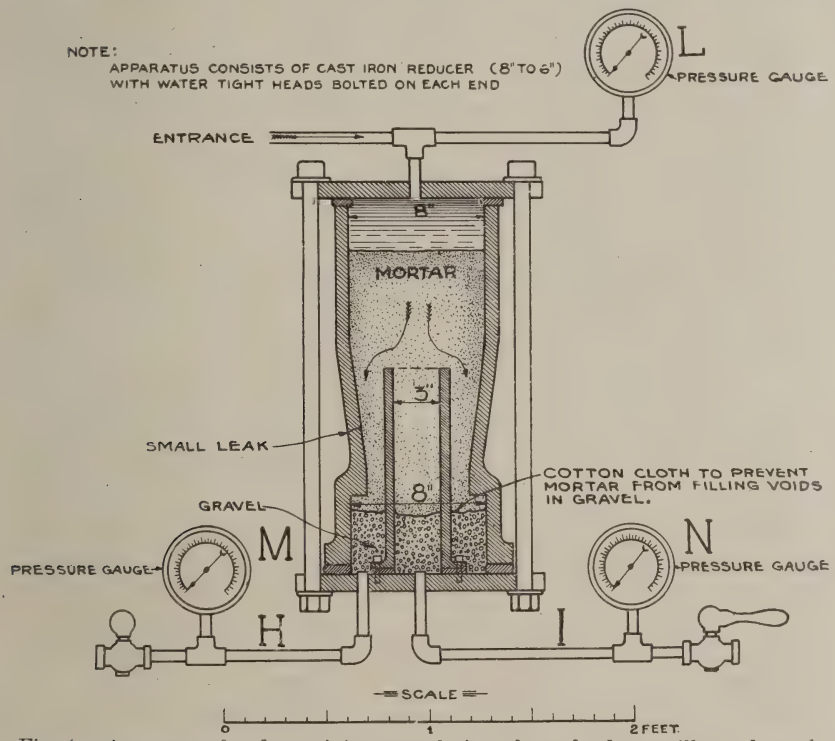

Fig. 1. Apparatus for determining percolation through the capillary channels in cement motar.

a. m. Pipes I and H both closed. Pressure at gauge M went up from 39.75 pounds to 73.5 pounds. Pressure at gauge N went up from 0 to 71.0 pounds. During the final three hours and eighteen minutes no change of pressure at either gauge.

Experiment 10: From April 23, 11.30 a. m., to April 24, 3.40 p. m. Pipe H open, pipe I closed. Pressure at gauge N went down from 71.0 pounds to 58.0 pounds. During the final seven hours and thirty minutes no change of pressure at gauge N.

Comment: These results show that water pressure is transmitted

through cement mortar. Mr. Francis advocates drains under dams.

17. During May and June, 1914, experiments were made at Dam No. 18, Ohio River, by Assistant Engineer Hall, under direction of Major Jervey, the District Engineer Officer at Wheeling, to determine pressure transmitted through concrete. A hole was drilled in river wall of lock as shown in Fig. 2. Such water as accumulated in the hole came by percolation and dropped down the sides or entered into the hole by pressure through the concrete. It was noticed that the rise was more rapid after the water had covered the wall for weeks than when wall had not been covered for some time and wall had dried out. The rate of rise of the water is shown in Fig. 2.

18. *Conclusion: Water pressure is transmitted through joints between concrete and concrete, and is transmitted more freely than through solid concrete. The water is transmitted quite slowly; and, if allowed free exit, the pressure is practically negligible.*

19. A hole was bored in river wall of lock at Dam No. 18, Ohio River, by Assistant Engineer Hall during May and June, 1914, to determine this also. The cross-section of wall, joint between concrete and water, and rate or rise of water are shown in Fig. 4.

20. *Conclusion: Water pressure is transmitted through joints between concrete and rock. This is generally transmitted more freely than through joints between concrete and concrete. The amount varies with different rocks. The water is transmitted quite slowly, in general, and if allowed free exit, the pressure is practically negligible in good rock without cracks or fissures.*

21. *Conclusion: Water travels in small veins in the joints between concrete and concrete, and between concrete and rock. If allowed free exit, the pressure varies irregularly between upper and lower pools; if exit is closed, the pressure quickly becomes that of upper pool, and water rises in test holes to level of upper pool as soon as enough water has passed through the veins to fill the test holes.*

22. *Conclusion: The amount of space of these small veins, that is, the area of upward water pressure, varies from nearly zero in excellent granite foundations to 50 per cent or more in rotten shale. It is practically impossible to calculate this. Some estimates could be made by calculating the rate of rise in test holes (see Figs. 4, 7, 8, 9 and 10) using head, length of travel, and size of test hole, but such calculated results would be very unreliable.*

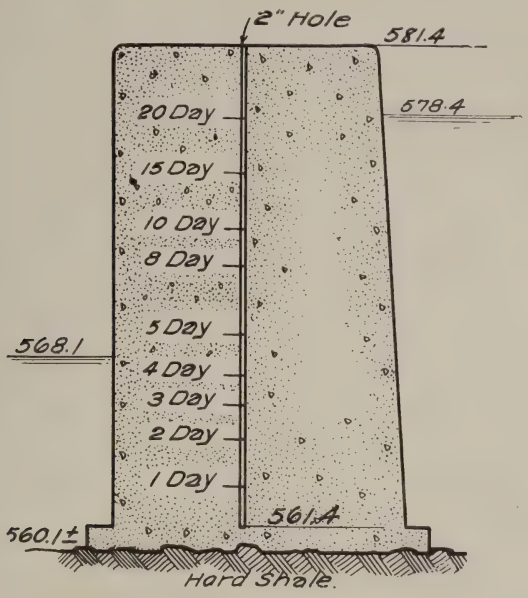
2" Hole
581.4
578.4
20 Day
15 Day
10 Day
8 Day
5 Day
568.1
4 Day
3 Day
2 Day
1 Day
561.4
560.1±
Hard Shale.

Fig. 2.

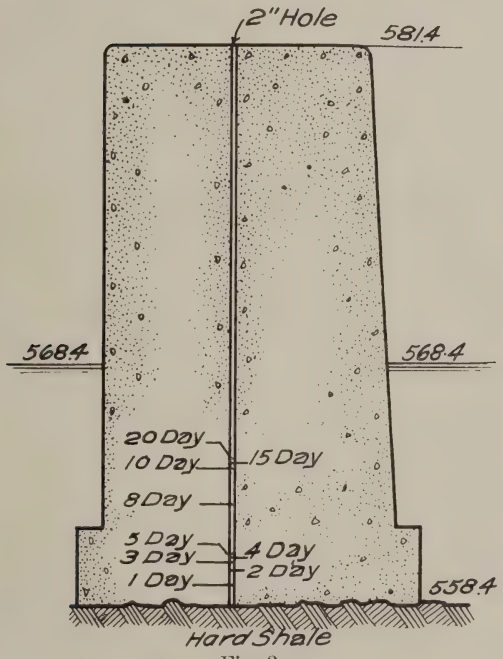
2" Hole
581.4
568.4
568.4
20 Day
10 Day
15 Day
8 Day
5 Day
3 Day
4 Day
1 Day
2 Day
558.4
Hard Shale

Fig. 3.

23. On page 247, Beardsley's "Hydro-electric Plants," we find the following statement:

Mr. J. B. Francis held that solid concrete deposited on bed rock would be lifted or floated, and to prove this, placed a pipe provided with a gauge in the concrete of a dam and found that the gauge registered the full pressure.

24. Mr. John D. Van Buren, in "Transactions of American Society of Civil Engineers," Volume XXXIV, advocated that a masonry dam be made strong enough to resist a pressure under its base equal to the full hydrostatic head in the reservoir.

25. In "Transactions of American Society of Civil Engineers," December, 1912, Volume LXXV, we find the subject for the first time given thorough consideration through a short paper by Mr. C. L. Harrison and a lengthy discussion by twenty-one other engineers, the whole occupying some eighty-four pages. Mr. Harrison enunciates three well-defined conditions as follows, and states his opinions as to the consideration of upward water pressure under these or intermediate conditions:

(1) In the case of a foundation of hard, sound rock, without either horizontal or vertical seams, there is no reason to expect that water will get into it and produce an upward pressure, and, in the design, no allowance should be made for it. In such cases the junction between the masonry and the foundation can easily be made water-tight.

(2) In the case where the foundation is stratified with well-defined horizontal seams, and the dam is located near a fall or rapids in the stream, so that the water may flow from the seams at the toe of the dam as freely as it enters them from the reservoir, the upward pressure will be approximately equal to the static head at the heel and gradually decrease to zero at the toe of the dam.

(3) Take a foundation similar to the foregoing in every respect except that the water in the seams of the rock can not escape freely near the toe of the dam, but must flow some distance downstream through rock or other materials before it reaches the surface of the ground, or must rise vertically to the surface: then the upward pressure at the heel will be equal to the static head, and that at the toe will be equal to the head required to overcome the resistance to the water escaping at that point.

Comment: All did not agree with these three conditions, and there was very much discussion with many calculations and figures, but few hard facts. In many cases the discussions are simply revisions of former articles or letters in engineering periodicals.

26. Mr. C. R. Weidner, of the University of Wisconsin, pub-

lished in the *Engineering News*, July 31, 1913, a paper on the subject of upward water pressure at masonry joints, quoting results of experiments made at the Oester and Nye dams in Germany. Concrete was the bonding material of the dam to the rock foundation, and the upward water pressures were in every case determined at the rock foundation; that is, between the concrete and the rock. The rock itself was graywacke and shale. The pipes were placed on the foundation and the height to which water rose in these pipes was measured as showing the amount of water pressure.

Comment: The records do not show that there was any lower pool, and the pressures shown in the piezometer pipes indicate no

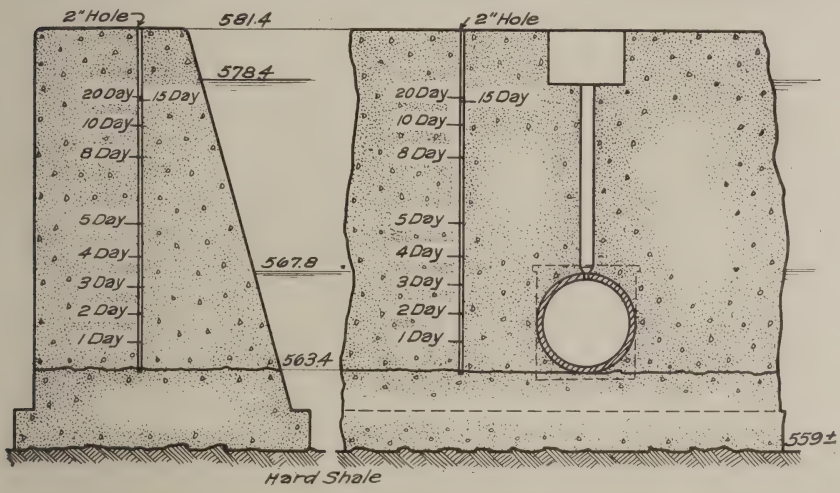

Fig. 4.

change of pressure because of the lower pool. Twenty-four of these piezometer tubes were used, and pressure existed in every one of them. The experiment showed that the piezometer pipes might be divided into two groups. In one group, comprising some five pipes, the water attained its final pressure in from nine to twenty-one minutes, showing that there was fairly free access of water to the foundation at these points. In the other tubes the water did not attain its final height until several hours and possibly some days.

27. Mr. C. M. Saville, Assistant Engineer on the Panama Canal, on pages 127-196, Report of the Isthmian Canal Commission, 1908, gives pressures found in test holes in rock, sand, and clay on the site of Gatun Dam. He found that the water remained at different heights in the different test holes, but at same height in same

test hole in practically all cases. He also found the rate of rise and fall of the water in these test holes, if water was removed or added, and noted that the water tended, in each hole, to return to a certain fixed height, this height varying in each hole; if water were poured in a hole it would gradually find its way out and level of water in the hole would fall; if water were pumped out of the hole, the hole would gradually fill itself. In general, these changes were rapid. These tests also showed the real height of the ground water. In this connection, Mr. Saville published figures of other dams showing the height of saturation (enforced saturation) in these dams already constructed (see figures published in *Engineering News,* November 28, 1901).

Comment: These are field tests, and are very valuable.

28. Three holes were left during construction (1909-1910) in the west land wall of Gatun locks in order to determine the upward water pressure. These holes were in the upper chamber wall, from about 50 feet from Gatun Lake to about 400 feet from the lake. The holes were not connected to each other, but were connected to drains under the lock floor, at same distance from Gatun Lake, 50 to about 400 feet, so that each hole shows resultant pressure of the combined pressures under the wall and the lock floor. The water rose very slowly and finally assumed definite levels at about 80 per cent of the level of Gatun Lake for the nearest hole, to 50 per cent for the farthest hole. A report of this experiment was submitted, but none has ever been published.

Comment: This result is about as expected, after studying experiments reported above by Mr. Weidner.

29. In the construction of the dam at Medina, Tex., Mr. C. H. Kearing left a drainage tunnel and seep holes as shown in Fig. 5. The foundation is a crystalline limestone, slightly porous, and free from seams. Water flows from the drain holes, but is diminishing somewhat. Systematic measurements have been taken of pressure in these seep holes. When pressure was measured in one hole all other holes were closed. The last tests available, over one year after completion, with upstream head of 113.65 feet above bottom of tunnel, show almost immediate pressures averaging 35 feet in parts of main tunnel and above 23 feet in cross tunnels.

Comment: In measuring these pressures, there was not enough delay to permit water to fill the corresponding vein throughout its

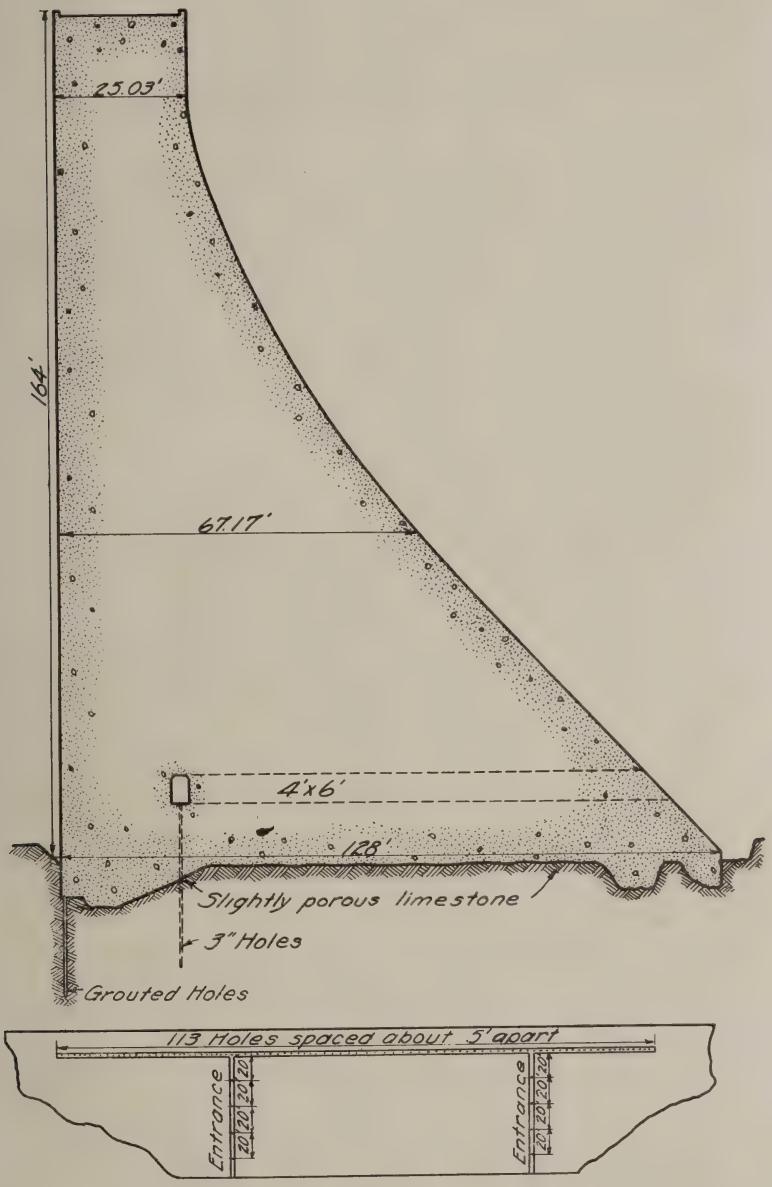
25.03'
164'
67.17'
4'x6'
128'
Slightly porous limestone
3" Holes
Grouted Holes
113 Holes spaced about 5' apart
Entrance
Entrance

Fig. 5.

length so as to give upper pool pressure. It seems probable that portions of the lengths of these veins are practically devoid of water, if there is free exit, because parts of the veins have greater slope or diameter than other portions, and water enters these portions less rapidly than it leaves; consequently, the full pressure can not be transmitted from the upper pool until the vein is full throughout.

30. During January and February, 1914, experiments were made at Lock No. 1, Monongahela River, by Assistant Engineer Fairchild under directions of Colonel Shunk, the District Engineer Officer at Pittsburgh, to determine actual upward water pressure in a horizontal joint between concrete and rock foundation. There were horizontal breaks in the monoliths; but a pipe was grouted in the full depth of each hole to prevent entrance of water into these holes by percolation through these horizontal joints. The concrete has been in position for several years.

The upward water pressures in the test holes, and the rate of rising to such water level, is shown in Figs. 6 and 7.

Comment: These experiments are actual field experiments, and show actual conditions. It is not easy to understand why the water in hole No. 2 should not rise to the level of the upper pool; probably there is a good drain or fissure leading to lower pool. Possibly, in this and other holes where water is lower at two months than at one month, there are good drains to lower pool and the lock chamber was at lower pool at the time when observation was taken.

31. During May and June, 1914, under the direction of Major Jervey, the District Engineer Officer at Wheeling, experiments were made at Dam No. 18 by Assistant Engineer Hall, and at Dam No. 26 by Assistant Engineer Ross, to determine pressure transmitted through joints between concrete and rock. Results showing cross-section, rate of rise, etc., are shown:

At Dam No. 18, Fig. 3.

At Dam No. 26, Figs. 8, 9, and 10.

32. *Conclusion: Water pressure is transmitted more or less freely through sand, gravel aggregate, and washed river gravel; and hardly at all through alluvial soil.*

33. During November and December, 1913, and January and February, 1914, many experiments to determine upward water pressure and percolation were conducted by Junior Engineer Barker and Inspector Ewell, under the direction of Major Jervey, the District Engineer Officer at Wheeling. The weather was cold and freezing part of the time, but care was taken and the weather had

Fig. 6.

no effect on the results. Only a few experiments are given in this paper.

34. The *apparatus* used was a wooden box, lined with galvanized iron, as shown in Fig. 11. It is intended to represent approximately a lock, river wall, etc., and conditions were made to represent those found in practice, except that in addition to normal heads of water excessive heads were used. The partition represents sheet piles and the concrete slabs represent the lock floor. The baffles were pieces of timber, of sizes shown in sketches, attached to the bottom and sides of the box to prevent free flow of water along these parts. A strip of wood, not shown in the sketches, was nailed on the inside along each side of the box to support the sides of the floor slab. In all cases the slabs were calked and made as near water-tight as possible.

The deeper part of the box formed a rectangular standpipe, so that a head of nearly 6 feet of water could be maintained above the overflow, which was about ½ inch lower than the top of the slab. Water was let in at the top of the standpipe and flowed through the sand, along the sheet piles, between the slabs and the bottom of the box, and out at the overflow. Leakage through box was practically nothing, though there was some leakage through calking around the slabs. This iron box was given a coat of paint on the inside and the paint was sanded to roughen the surface and prevent free flow along iron bottom. In all cases the materials to be tested were rammed or puddled in water to represent actual conditions.

35. *Explanation of nomenclature.* All tube readings are in inches above overflow. This varies slightly, but practically means pressure at level of top of slab.

"Sand" means washed river sand screened through a ¼-inch screen.

"Washed gravel" means ordinary river gravel washed and screened through a 2-inch, and retained on a ¼-inch, screen.

"Gravel aggregate" means ordinary river-bottom mixture of gravel, sand, and mud at the site of Dam No. 14, without any washing or screening whatever.

"Alluvial soil" means ordinary sedimentary deposit on the banks of the river, as was obtained from the field near the apparatus.

36. Experiment—

Material: Sand (with free exit), Fig. 11.

Object: To determine upward water pressure.

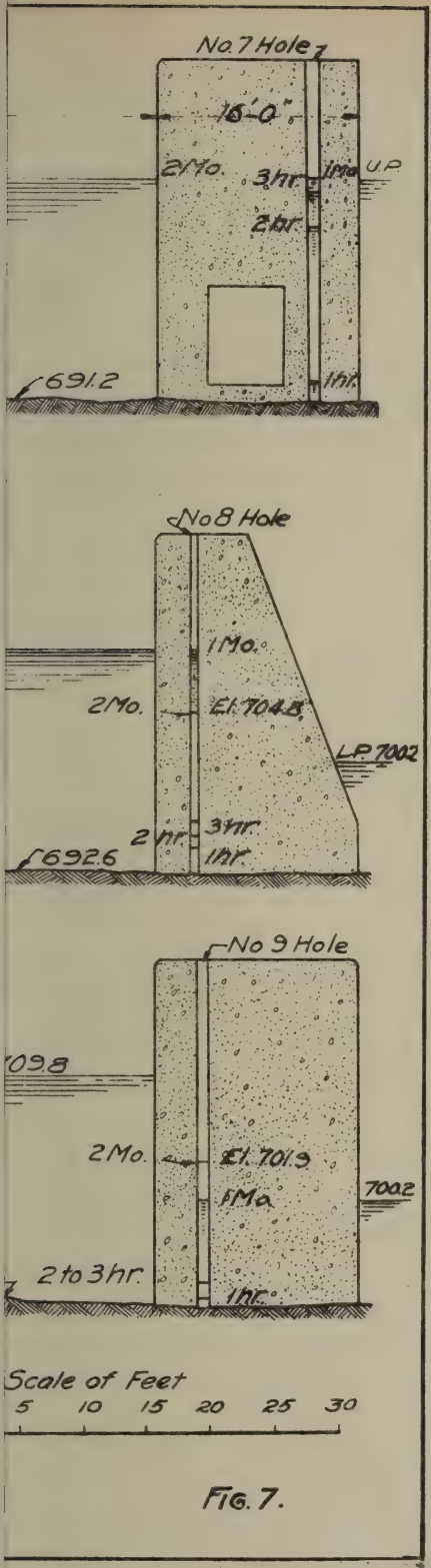

Fig. 7.

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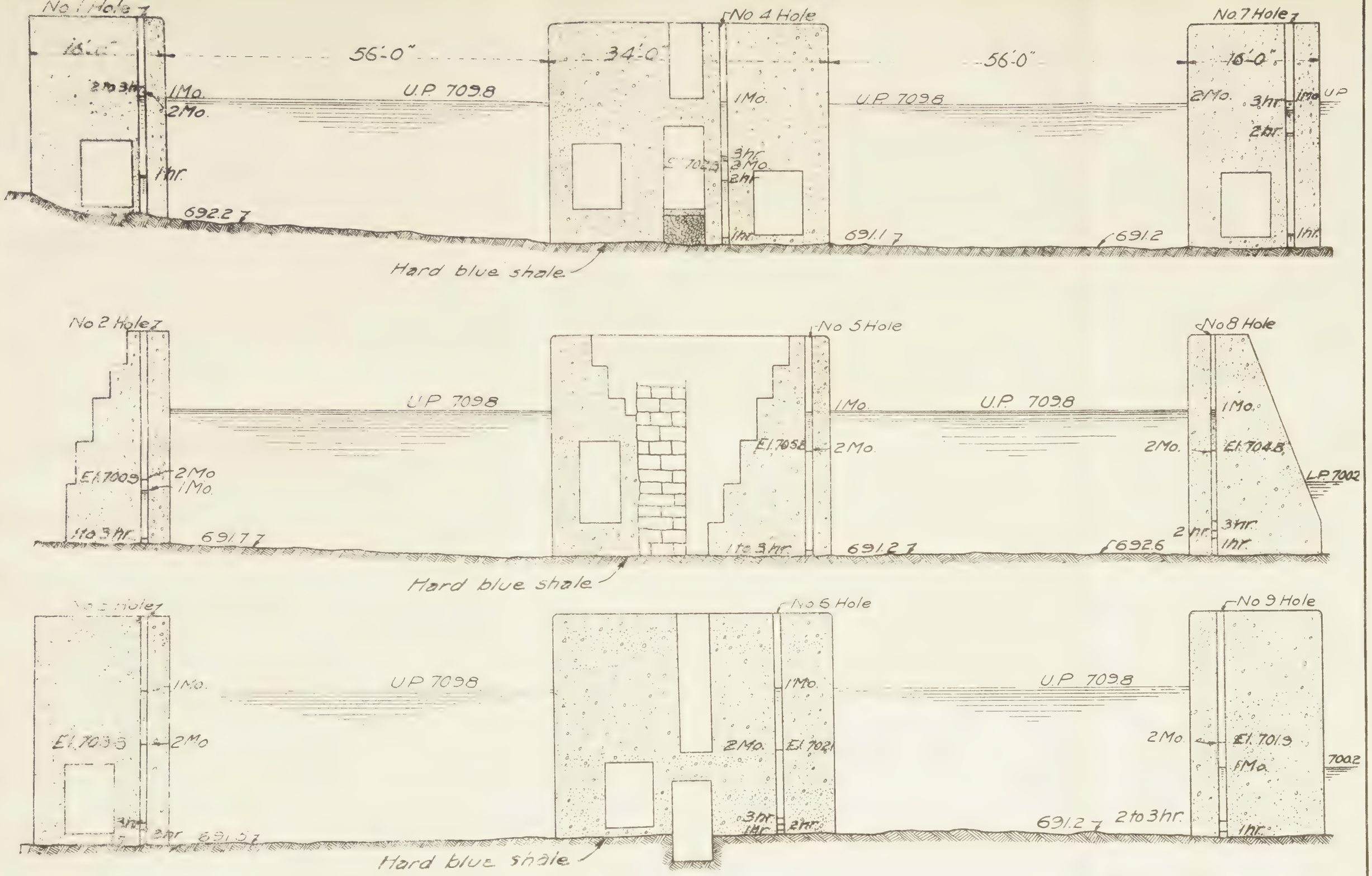

MONONGAHELA RIVER.
LOCK 1
Cross-Sections of Lock Walls showing Test Holes.

FIG. 7.

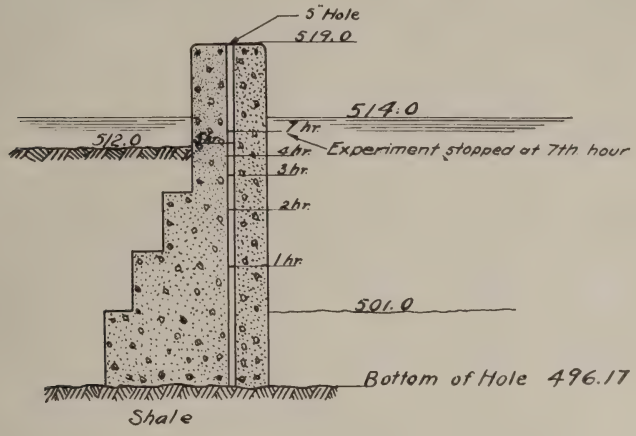

Fig. 8.

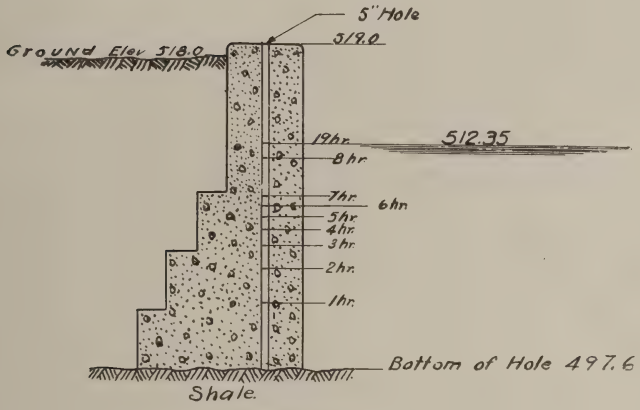

Fig. 9.

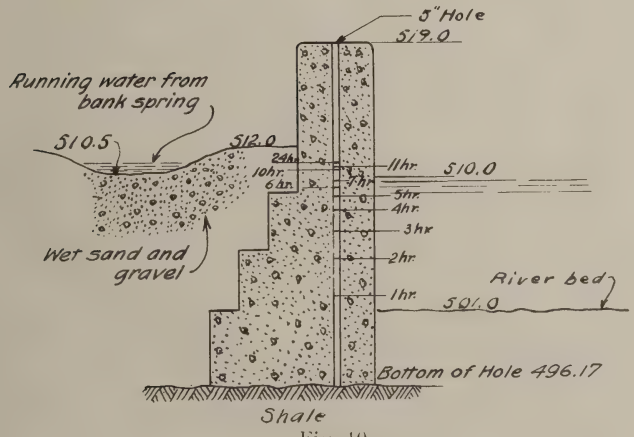

Fig. 10.

Conditions: Nine concrete slabs were in place; the overflow was open.

Results: The upward water pressure under foundation is not at all that of the head. (Fig. 14.)

Head.	Head held.	Water in tubes above concrete.								
		A	B	C	D	E	F	G	H	I
Inches.										
16	0 min.	----	----	----	----	----	----	----	----	----
16	65 do.	$5\frac{3}{4}$	$5\frac{3}{4}$	$5\frac{3}{4}$	$5\frac{3}{4}$	$5\frac{1}{2}$	5	5	$4\frac{1}{2}$	4
30	0 ris'g	$10\frac{1}{4}$	$10\frac{1}{4}$	$10\frac{1}{4}$	$10\frac{1}{4}$	$9\frac{3}{4}$	$8\frac{3}{4}$	$8\frac{1}{2}$	$7\frac{3}{4}$	7
42	do.	12	$11\frac{3}{4}$	$11\frac{3}{4}$	$11\frac{3}{4}$	11	10	$9\frac{1}{2}$	9	$7\frac{3}{4}$
48	do.	$12\frac{1}{4}$	$12\frac{1}{4}$	$12\frac{1}{4}$	$12\frac{1}{4}$	$11\frac{1}{4}$	$10\frac{3}{4}$	10	$9\frac{1}{4}$	$8\frac{1}{4}$
49	do.	----	----	----	----	----	----	----	----	----

At this point a boil occurred at the edge of slab I before the heads could be read.

37. Experiment—

Material: Gravel aggregate (with free exit), Fig. 11.

Object: To determine upward water pressure.

Conditions: Nine slabs were in place; the overflow was open.

Results: The upward water pressure varied from one-sixteenth the head at the heel to one-fortieth the head at the toe (Fig. 14).

Head.	Head held.	Water in tubes above concrete.								
		A	B	C	D	E	F	G	H	I
Inches.										
18	0 ris'g	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$
36	do.	2	$1\frac{7}{8}$	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{1}{4}$	1	1	$\frac{7}{8}$	$\frac{3}{4}$
60	do.	$3\frac{3}{8}$	3	$2\frac{7}{8}$	$2\frac{3}{4}$	$2\frac{1}{4}$	2	$1\frac{7}{8}$	$1\frac{3}{4}$	$1\frac{3}{8}$

38. Experiment—

Material: Washed gravel (with free exit), Fig. 11.

Object: To determine upward water pressure.

Conditions: Nine concrete slabs were in place; the overflow was open.

Results: The upward pressure varied from full upward pressure at the heel to about one-third at the toe (Fig. 14).

Head.	Water in tubes above concrete.								
	A	B	C	D	E	F	G	H	I
Inches.									
$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	1	1	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{5}{8}$
$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{3}{8}$	2	$1\frac{5}{8}$	$1\frac{5}{8}$	$1\frac{1}{2}$	1	$\frac{3}{4}$

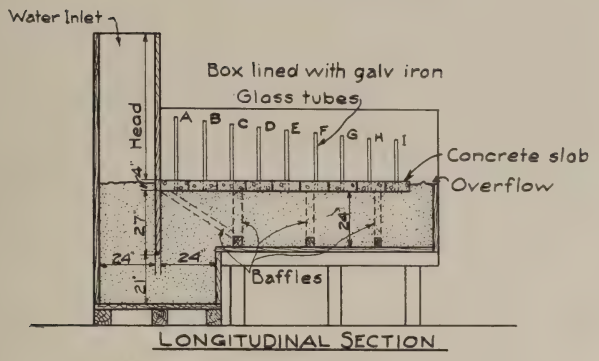

Fig. 11.

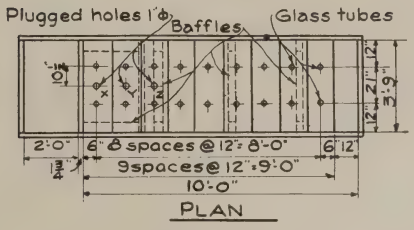

Fig. 12.

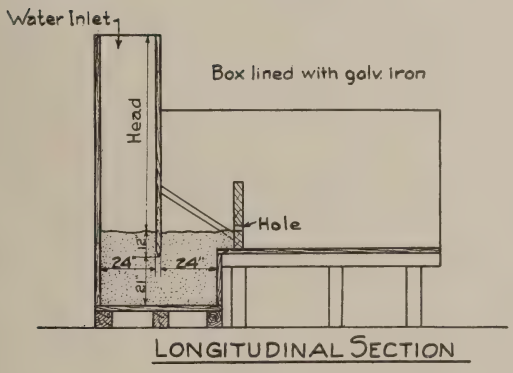

Fig. 13.

The 2½-inch head was the greatest that could be maintained with the available water supply; water percolated more rapidly through the washed gravel than it entered through the supply pipe.

39. EXPERIMENT—

Material: Alluvial soil (with free exit), Fig. 11.

Object: To determine upward water pressure.

Conditions: One concrete slab next to sheet piles. The soil was placed in the entire box to the elevation of the bottom of the slab.

Results: No upward water pressure was obtained under these conditions.

Head.	Head held.	Tube reading (A).
Inches.		
21	0	0
36	1 hr. 25 min.	0
48	1 hr.	0
60	1 hr. 20 min.	0
64	3 hrs.	0

Remarks: A *very* slight percolation, but no upward pressure; upward pressure might have been obtained, if experiment had been continued several days.

40. EXPERIMENT—

Material: Sand (without free exit), Fig. 11.

Object: To determine upward water pressure.

Conditions: Nine concrete slabs were in place, and the overflow closed. Leakage from the box was practically zero through calking around slabs.

Results: Upward water pressure was practically one-half the head when slabs rose.

Head.	Head held.	Water in tubes above concrete.								
		A	B	C	D	E	F	G	H	I
Inches.	*Min.*									
11	0	----	----	----	----	----	----	----	----	----
11	50	5	5	5	5	5	5	5	5	5
23	0	----	----	----	----	----	----	----	----	----
23	25	13¾	13¾	13¾	13¾	13¾	13¾	13¾	13¾	13¾
30	0	----	----	----	----	----	----	----	----	----
30	10	15¾	15	15	15	15	15	15	15	15
30	40	16	15	15½	15½	15½	15	15½	15½	15½
40	0	----	----	----	----	----	----	----	----	----
40	10	16¼	16¼	16¼	16¼	16¼	16¼	16¼	16¼	16¼

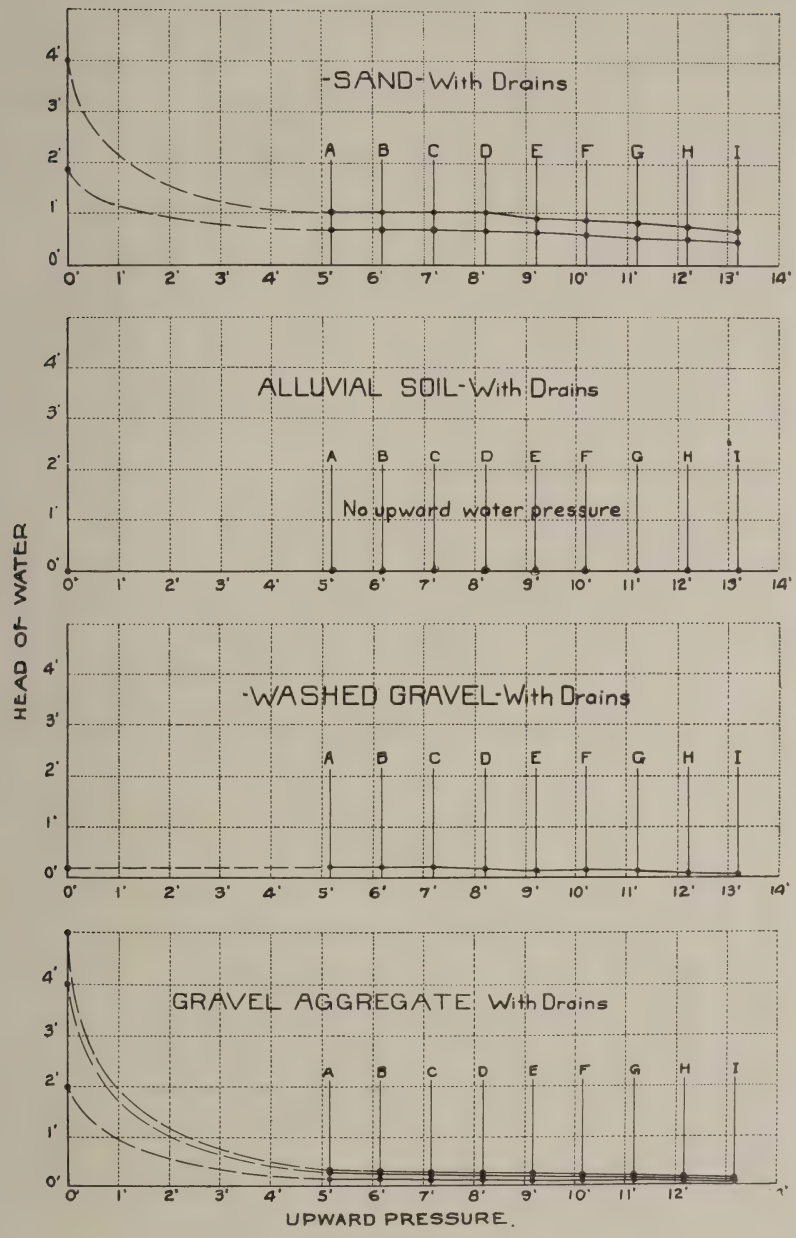
-SAND- With Drains
ALLUVIAL SOIL- With Drains
No upward water pressure
-WASHED GRAVEL- With Drains
GRAVEL AGGREGATE With Drains
A B C D E F G H I
0' 1' 2' 3' 4' 5' 6' 7' 8' 9' 10' 11' 12' 13' 14'
HEAD OF WATER
UPWARD PRESSURE.

Fig. 14.

At this point the slabs E, F, G, and H started to rise in the box, and the experiment was stopped when the slabs F and G were 1 inch above the supporting strips. As a head of 9 inches of water should have lifted these slabs, it is thought that the friction of the calking was equal to the other 7 inches. This was not measured, but seems reasonable in view of the strength exerted by laborer to remove the slabs.

41. Experiment—

Material: Washed gravel (without free exit), Fig. 11.

Object: To determine effect of a few relief holes on upward water pressure on slab.

Conditions: A bulkhead was used 3 feet out from the sheet piles. Gravel was placed to the elevation of the bottom of the slab. Three slabs were placed. Each of these three slabs had a hole 1 inch in diameter in its center.

Results: With heads of about 1 foot and the three 1-inch holes closed, the upward water pressure was equal to the head. When the holes were suddenly opened the upward water pressure decreased about 0.2 of head. It is evident that the three relief holes could not carry away the water as fast as it percolated through the washed gravel.

Tube.	Head with holes closed.	Head with holes suddenly opened.	Difference.
	Inches.	*Inches.*	*Inches.*
B	$11\frac{3}{4}$	$9\frac{1}{4}$	$2\frac{1}{2}$
C	$11\frac{1}{2}$	$8\frac{3}{4}$	$2\frac{3}{4}$
A	11	$8\frac{1}{2}$	$2\frac{1}{3}$

42. Experiment—

Material: Gravel aggregate (without free exit), Fig. 11.

Object: To determine the effect of a few relief holes in a concrete slab, as to influence on upward water pressure.

Conditions: A bulkhead was used 3 feet out from the sheet piles. The aggregate was placed to the elevation of the bottom of the slab on both sides of the sheet piles, and three concrete slabs were placed. Slabs *A* and *C* each had a hole 1 inch in diameter in the center, referred to as "X" and "Z."

Results: (*a*) With all exits closed, the upward pressure was equal to the head; (*b*) with one relief hole open, the upward water pressure was about one-eighth the head; (*c*) with two relief holes open, the upward pressure was about one-twentieth the head.

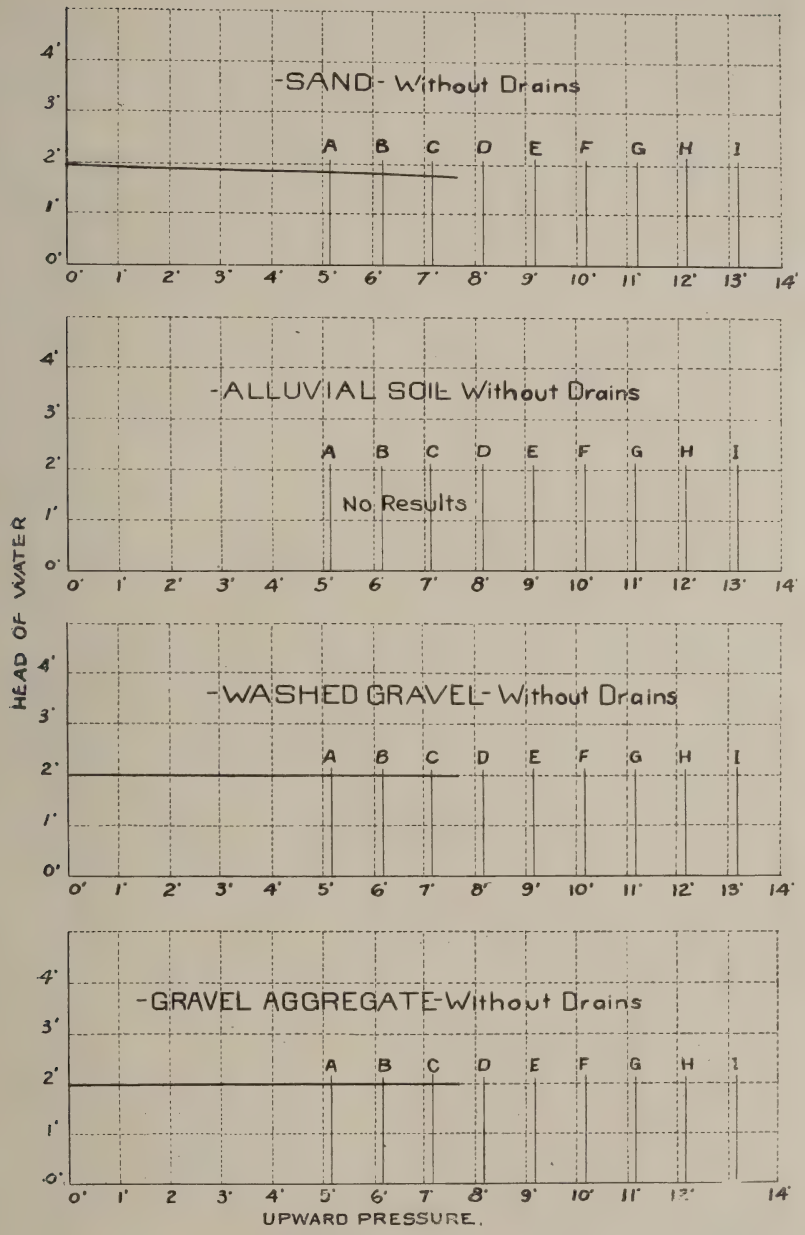
-SAND- Without Drains
-ALLUVIAL SOIL Without Drains
No Results
-WASHED GRAVEL- Without Drains
-GRAVEL AGGREGATE-Without Drains
A B C D E F G H I
0' 1' 2' 3' 4' 5' 6' 7' 8' 9' 10' 11' 12' 13' 14'
0' 1' 2' 3' 4'
HEAD OF WATER
UPWARD PRESSURE.

Fig. 15.

(*a*) With "X" and "Z" closed, the results were as follows:

Head.	Head held.	Tubes.		
		A	B	C
Inches.	*Minutes.*	*Inches.*	*Inches.*	*Inches.*
12	10	12	12	12
22	20	22	22	22

(*b*) With "X" open and "Z" closed, the results were as follows:

Head.	Head held.	Tubes.		
		A	B	C
Inches.		*Inches.*	*Inches.*	*Inches.*
22	Rising	$2\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{3}{4}$
40	do.	5	$5\frac{1}{2}$	6
64	do.	$9\frac{1}{2}$	$10\frac{1}{4}$	$10\frac{1}{4}$

(*c*) With "X" and "Z" open, the results were as follows:

Head.	Head held.	Tubes.		
		A	B	C
Inches.		*Inches.*	*Inches.*	*Inches.*
64	0 Falling	$3\frac{1}{4}$	$2\frac{1}{2}$	$1\frac{1}{2}$
40	do.	$1\frac{3}{4}$	$1\frac{1}{4}$	1
22	do.	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{1}{2}$

43. EXPERIMENT—

Material: Sand (without free exit), Fig. 13.

Object: To determine if full head will be transmitted through the sand.

Conditions: Sand was placed as shown in Fig. 13; outlet rendered practically water-tight; drain hole stopped up. Only one head was used—2 feet.

Results: It is believed that full head will be obtained in less than twenty-four hours if exit is made absolutely water-tight. (Fig. 15.)

Head.	Height of water above level of sand.	Notes.
Feet.	*Inches.*	
2	0.0	6.00 a. m.—started.
	9.0	8.00 a. m.
	11.5	10.00 a. m.
	14.0	12.00 m.
	16.0	2.00 p. m.
	17.5	4.00 p. m.
	18.5	6.00 p. m.
	19.0	8.00 p. m.
	19.25	10.00 p. m.
	19.75	6.00 a. m.

Leakage through bulkhead in box at this head (19.75 inches) was probably equal to percolation.

44. EXPERIMENT—

Material: Gravel aggregate (without free exit), Fig. 13.

Object: To determine if full head will be transmitted through the gravel aggregate.

Conditions: Gravel aggregate was placed as shown in Fig. 13; outlet rendered practically water-tight; drain hole stopped up. Only one head was used—2 feet.

Results: Full head was obtained. (Fig. 15.)

Head.	Height of water above gravel aggregate.	Notes.
Feet.	*Inches.*	
2	0.0	10.25 a. m.—started.
	12.0	11.00 a. m.
	18.5	11.25 a. m.
	23.5	12.25 p. m.
	24.0	2.25 p. m.

45. EXPERIMENT—

Material: Washed gravel (without free exit), Fig. 13.

Object: To determine if full head will be transmitted through the washed gravel.

Conditions: Washed gravel was placed as shown in Fig. 13; outlet rendered practically water-tight; drain hole stopped up. Only one head was used—2 feet.

Results: Water ran through gravel so rapidly that the head of 2 feet was reached almost simultaneously on both sides. (Fig. 15.)

Head.	Total hours head held.	Height of water above level of wash gravel.	Notes.
Feet.		*Feet.*	
2	------	2	Water ran through gravel as fast as apparatus supplied it.

46. *Comments:* These experiments were carefully conducted, and the results are considered valuable. Further conclusions are as follows:

47. *Conclusion: Puddled alluvial soil has practically no upward water pressure, if there are any drains; but such soil quickly disintegrates with a running leak.*

With a material such as alluvial soil, which offers so great resistance to percolation as to practically prevent it, a very small number of relief holes in the slab would be sufficient to practically eliminate the upward water pressure, provided, of course, that the material were such that only percolation and no erosion took place.

48. *Conclusion: With a material offering so great a resistance to percolation as this sand, a very small number of relief holes will greatly reduce the upward water pressure when there is no outlet at the toe of the slab.* In this case the one open hole reduced the upward water pressure to one-twentieth the head.

With a slab having free exit for water at the toe, and a row of sheet piles at the heel, the upward water pressure varies from one-third the head at the heel to one-fourth the head at the toe.

49. *Conclusion. This gravel aggregate offers less resistance to percolation than the river sand, and more than the river gravel. Hence, for the same reduction of upward water pressure when there is no outlet at the toe of the slab, more relief holes will be needed than for the sand and less than for the washed gravel.* In this case the one open hole reduced the upward water pressure to one-eighth the head.

For a slab having free exit for water at the toe and a row of sheet piles at the heel, the upward water pressure varies from one-sixteenth the head at the heel to one-fortieth the head at the toe.

50. *Conclusion: In a material offering such a small resistance to percolation as this washed gravel, a large number of relief holes in the slab will be necessary to materially reduce the upward water pressure when there is no outlet at the toe of the slab.*

With an outlet for water at the toe of the slab and a row of sheet piles at the heel, the upward water pressure varies from full head at the heel to about one-third the head at the toe.

51. *Conclusion: The pressure of water with free exit, transmitted through sand, washed gravel, or gravel aggregate, is greatly diminished on entrance and more slowly, but surely, diminished along line of travel* (see Fig. 14).

52. *Conclusion: Full upward pressure of water, without free exit, is transmitted quickly through washed gravel, and less quickly through gravel aggregate and sand.* (See Fig. 15.)

53. Others have reached conclusions somewhat similar to the above; as is seen from the following:

(*a*) Mr. W. C. Hammatt, of San Francisco, Cal., published in the *Engineering News* on October 30, 1913, certain investigations which he had made concerning percolation on canals. Fig. 17 shows "Parallelism between flow of water in pipes and flow of seepage water from canals."

Comment: The only point of interest bearing on the subject of this paper is the evident fact that the pressure of the water de-

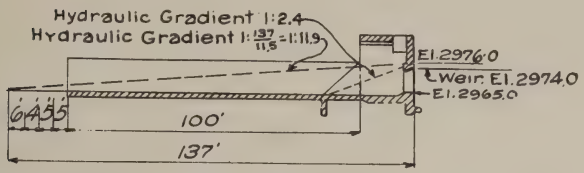

Fig. 16. Cross-section through intake showing hydraulic gradient and piezometric line.

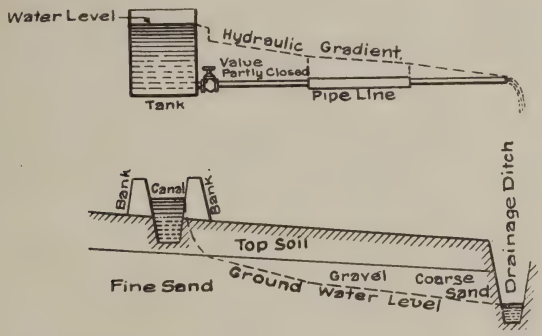

Fig. 17. Parallelism between flow of water in pipes and flow of seepage water from canals.

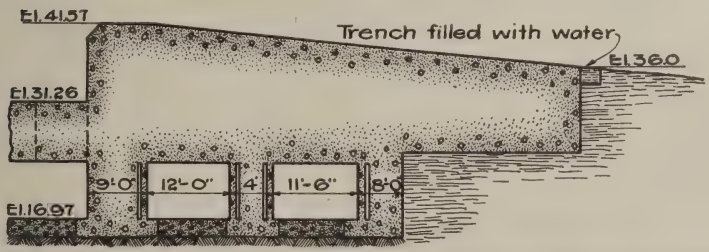

Fig. 18. Section through shell room and magazine of 12-inch gun emplacement, Fort Armistead, Md.

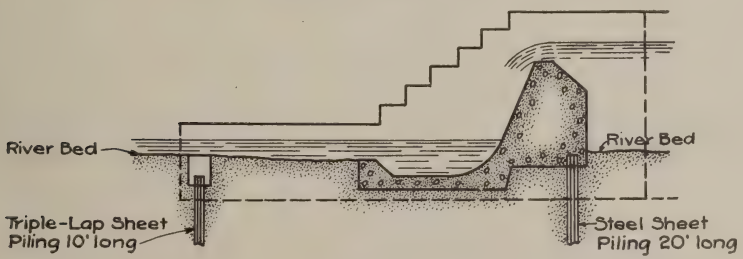

Fig. 19.

creases a very great deal on leaving the canal, and thereafter decreases more slowly. This would correspond to an actual condition where the water left a tank by way of a valve partly closed. The above result corresponds to results of experiments by Ohio River Board. (See Fig. 14.)

(*b*) Mr. G. E. P. Smith, on page 193, "Transactions of the American Society of Civil Engineers," Volume LXXIII, September, 1911, discusses a paper on dams on sand foundations by Mr. Arnold C. Koenig.

Fig. 19 shows a cross-section of a dam which is stated by Mr. Koenig as being satisfactory for a typical design.

Fig. 20 shows the pressures on this dam as computed by Mr. Smith.

Fig. 21 shows another method by Mr. Smith of computing these pressures, giving practically the same conclusions as to the locality and force of the pressures.

Comment: It will be noted that Mr. Smith assumes the pressure to vary with the length of travel of percolating water, and finally has a pressure of zero at the end of the travel.

PERCOLATION.

54. Percolation is the cause of the upward water pressure. Consequently, the preceding experiments are to be considered in connection with percolation. The conclusions herewith are based on these and other experiments described below.

55. *Conclusion: Water percolates (very slowly) through concrete and rock.*

56. Mr. David M. Francis, past president of the American Society of Civil Engineers, in October, 1888, read before the American Society of Civil Engineers a paper giving results of experiments, with apparatus shown in Fig. 1, to determine percolation through Portland cement mortar, mixture 1 to 2. He took great precautions to prevent leakage. The following are his results:

Experiment 1: From April 12, 10.55 a. m., to April 12, 1.58 p. m. Pipe H closed, pipe I open. Pressure at gauge L, 77.0 pounds. Pressure at gauge M went up from 27.0 pounds to 64.0 pounds. Drip from pipe I in 183 minutes, 1.453 pounds. Rate of percolation in twenty-four hours:

$$\frac{1440 \times 1.453}{183} = 11.433 \text{ pounds.}$$

Experiment 2: From April 24, 3.53 p. m., to April 25, 3.53 p. m. Pipes I and H both open. Pressure at gauge L, 77.0 pounds. Drip from pipe I in the twenty-four hours, 8.256 pounds. Drip from

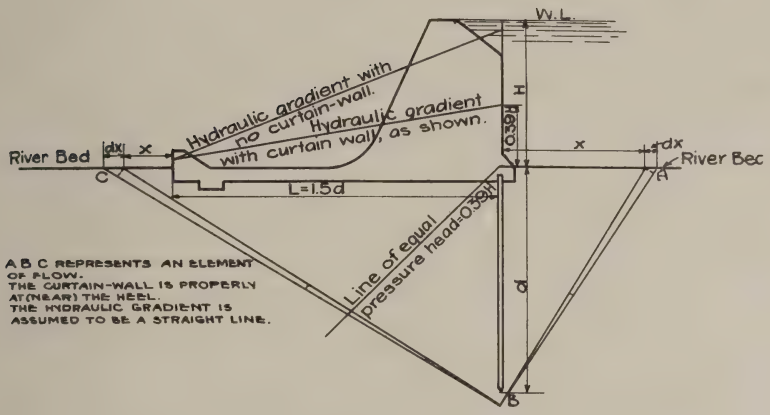

Fig. 20. An approximate analysis of the seepage flow beneath a dam or other structure on a pervious bed.

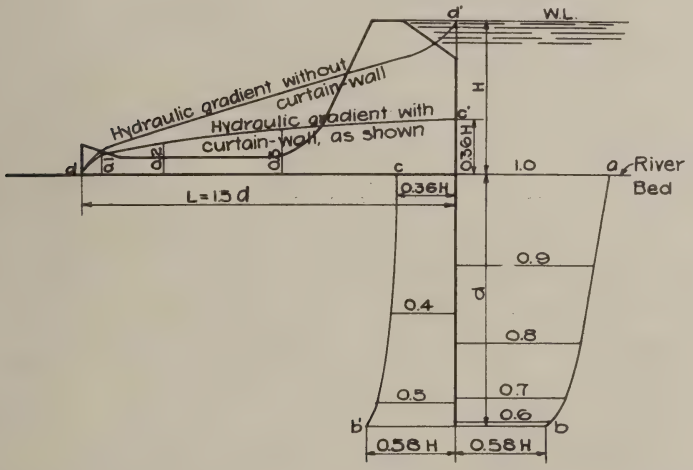

Fig. 21. A study of the amount and distribution of pressure on both sides of a curtain-wall and on the base of a dam or other structure on a pervious bed. Base of dam = 1.5 depth of curtain-wall.

pipe H in the twenty-four hours, 1,762 pounds. Total amount of percolation in the twenty-four hours, 10.018 pounds.

Comment: These experiments show that there is some percolation through cement mortar even under laboratory conditions.

57. On page 2461 of the Report of the Chief of Engineers for 1902, recopied in *Engineering News* for April 2, 1903, we find the following report of an experiment by Col. Peter C. Hains, Corps of Engineers, showing that water easily seeps through concrete:

To demonstrate the permeability of concrete, an experiment was made at the 12-inch gun magazine at Fort Armistead, near Baltimore, Md. A trench 4 feet wide, 4 feet deep and 30 feet long was dug in the earth parapet at the edge of the concrete in front of this magazine, as shown by Fig. 18. This trench was filled with water and on the second day water commenced dripping from the roof of the powder magazine and shortly after from both rooms, running down the side walls. There had been no rain for a month, and the magazines, before the water was put into the trench, were comparatively dry. This showed conclusively that there was less resistance to the passage of water through the 30 feet of concrete than to its passage through the sandy material forming the earthen portion of the parapet. To shut off the water from these rooms and from similar rooms at Fort Howard and Battery Stricker, inside metal ceilings were built which drained to the air spaces at the sides of the rooms.

Comment: This is practically the same result as that obtained by Mr. Francis.

58. Mr. C. M. Saville, Assistant Engineer on the Panama Canal, on pages 127-196, Report of the Isthmian Canal Commission, 1908, gives results of permeability tests on the various kinds of rocks found at the site of Gatun Dam. His experiments were very carefully conducted and very thorough for that site. He concludes that practically all rocks are more or less permeable. He states, what is generally believed, that the loss in head varies directly as the length of travel.

In this same report, Mr. Saville made experiments as to the porosity and seepage of the various soils, sand and clay, found at the site of Gatun Dam. On page 153, he shows the actual amounts of filtration through fixed area with a fixed loss of head in travel through various kinds of sand.

Comment: These are real field experiments, and show that all rocks are more or less permeable and that there is seepage in practically all soils.

59. Major Jervey at Wheeling made many experiments, a few

of which are given below, to determine rate of percolation with various materials.

60. *Conclusion: There is very little percolation through alluvial soil.*

61. EXPERIMENT—

Material: Alluvial soil, Fig. 11.

Object: To determine rate of percolation.

Conditions: The soil was first placed to the elevation of the bottom of the slab; then, as no percolation was obtained, layers were removed from the "downstream" side of the sheet piles, and later the thickness on the "upstream" side was decreased.

Results: Practically no percolation; finally blow-out.

Head.	Time standing.	Length of travel for percolation.	Percolation.
5 ft. 6 in.	40 min.	48 in.	
6 ft. 0 in.	12 hrs.	42 in.	3 gal. per hr.
6 ft. 6 in.	2 hrs.	36 in.	
6 ft. 9 in.	2 hrs.	33 in.	
7 ft. 0 in.	1 hr. 40 min.	30 in.	
7 ft. 3 in.	42 hrs.	27 in.	3 gal. per hr.
7 ft. 11 in.	1 hr. 10 min.	22 in.	
7 ft. 11 in.	1 hr. 15 min.	16 in.	8 gal. per hr.
7 ft. 11 in.	2 min.	9 in. blow-out	

62. *Conclusion: Water percolates more of less freely through sand, gravel aggregate, and washed gravel.*

63. EXPERIMENT—

Material: Sand, Fig. 13.

Object: To determine rate of percolation.

Conditions: Sand was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet.

Results: Percolation slow but constant.

Head.	Head held.	Percolation per minute.
Feet.	*Hours.*	*Gallons.*
1	24	0.125
2	24	0.3
3	24	0.8
4	24	1.1
5	24	2.8
6	3	9.0

64. EXPERIMENT—

Material: Gravel aggregate, Fig. 13.

Object: To determine rate of percolation.

Conditions: Gravel aggregate was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet.

Results: Percolation much more rapid than through sand, but much less than through washed gravel.

Head.	Head held.	Percolation per minute.
Feet.	*Hours.*	*Gallons.*
1	24	3.2
2	22	6.7
3	8	11.0
4	4	21.6
5	4	22.3
$5\frac{3}{4}$	$\frac{1}{2}$	42.0

65. Experiment—

Material: Washed gravel, Fig. 13.

Object: To determine rate of percolation.

Conditions: Washed gravel was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet.

Results: Percolation very rapid.

Head.	Head held.	Percolation per minute.
Inches.	*Hour.*	*Gallons.*
4	1	About 25

66. *Conclusion: Water creates "boils" at the toe of the foundation, after the relation of head to length of travel (percolation factor) becomes a certain amount, varying for different materials. This condition is dangerous to the foundation when the velocity of exit is sufficient to carry away with it the material of the foundation. It is generally best to make this percolation factor so great that there will be no such boils.*

67. *Conclusion: If the percolation factor is very small (varies for different materials), the material under the foundation will blow out and the foundation will fall in. With alluvial soil, this blow-out comes with little or no previous warning by boils.*

68. Mr. W. G. Bligh, Toronto, Canada, on December 29, 1910, published in the *Engineering News* a paper on "Dams, Barrages, and Weirs on Porous Foundations." Mr. Bligh defines percolation factor as the quotient of the length of travel of the water divided by the head of water. He discussed various conditions and

reached the conclusion that the following are suitable percolation factors for safety in construction:

Fine silt and sand, as in Nile River	18
Fine micaceous sand, as in Colorado and Himalayan rivers	15
Ordinary coarse sand	12
Gravel and sand	9
Boulders, gravel and sand	4 to 6

As these percolation factors would necessarily require a dam with a very wide base as compared with its height, Mr. Bligh devised certain schemes for reducing this length of travel of water; that is, for reducing the width of the base he recommended sheet piles, or cut-off walls, down under the material of the dam and considered the travel down one side and up the other side of the sheet piles, or cut-off wall, as being of practically equal value for length as the corresponding width of dam.

He gives several instances of dams which were safe with a less percolation factor than that of dams which blew out. From this it is seen that the percolation factor varies with different foundations.

For example, the Narrora weir on the Ganges had a percolation factor of about 11 (see Fig. 22). It stood in this condition for about twenty years, but the rear apron was washed out and the percolation factor fell to 9½, at which time the whole floor was *blown up.* As reconstructed and now standing, it has a percolation factor of 15.7 (see Fig. 23). In another case, the Chenab River weir, with a percolation factor of about 8.3 was undermined, and the present reconstructed weir with a percolation factor of 16 is now standing. Mr. Bligh shows several drawings of various dams, and says that in rotten silt and sand percolation factors as high as 15 should be the rule and not the exception.

Comment: On page 108, River and Canal Engineering, Bellasis, it is stated: "For small works, . . . steeper gradients are permissible."

69. Mr. Bligh, on April 13, 1911, published in the *Engineering News* another paper on the same subject. In this paper he brought out the fact that the expensive curtain walls, or sheet piling, extending below the dam, could be abolished. Naturally, this would decease the length of travel of the water, and, consequently, would decrease the percolation factor; but he showed a scheme whereby the base could be made correspondingly wider at less expense than with these curtain walls or sheet piling. Fig. 24 shows proposed

design for a typical section of rock-filled weir with intermediate vertical walls through rock fill. This pressure, due to head of water, is decreased in travelling down and under and up the far side of any of these walls. If the height of the next intermediate wall is such that it is equal to the head remaining, then the water will have to travel the full distance of the fill between the two walls and under and up the far side of the second wall before there will be any possibility of its escaping. If, again, a third wall is placed with a height equal to this remaining head, the length of travel is again increased and head decreased. By continuing to place these walls with decreasing heights, it is evident that all head of water can finally be eliminated. As the intermediate vertical walls are small and the space between is filled with riprap, it is seen that this construction is fairly cheap. The riprap has value also in that it prevents too easy egress of sand and packs the sand by adding a certain amount of weight, thereby decreasing still further the tendency of the water to percolate.

Fig. 25 shows the Okhla weir across the Jumna River in India, which was constructed in accordance with this idea. This has a percolation factor of about 15. It is not evident how the last 90 feet of the weir really assists in preventing blow-outs, except in so far as this riprap prevents scour, has weight, and packs the sand to a certain extent.

Fig. 26 shows the Laguna weir across the Colorado River. In this case the concrete floor on the top of the riprap is of value in preventing scour and possibly would hold down certain excessive water pressure by its own strength, but the essential principles remain the same as in the two preceding designs. Probably, the ideal weir, in accordance with this scheme, is given in Fig. 27, which shows the Madaya weir across the Madaya River in Burmah. It has a percolation factor of about 12, and the riprap at the foot of the slope is short in proportion to the rest of the dam.

Comment: These designs are certainly safe enough.

70. Mr. Bligh, on February 6, 1913, published in the *Engineering News* a paper on dams on sand or similar foundations, showing lessons learned from failure of a dam (name of dam not given). The cause of the failure was evident. A fore-apron, 100 feet long and 1 foot thick, formed part of the intake (see Fig. 16). Counting this forebay, the hydraulic gradient was 1 to 12, but not counting this forebay the hydraulic gradient was 1 to 2.4. The water

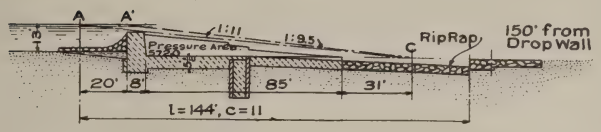

Fig. 22. Narrora weir as originally built.

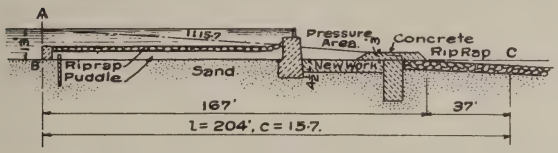

Fig. 23. Narrora weir as reconstructed.

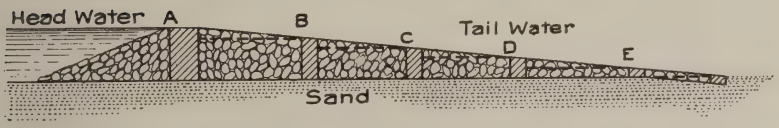

Fig. 24. Typical section of rock fill weir with intermediate vertical walls through rock filling.

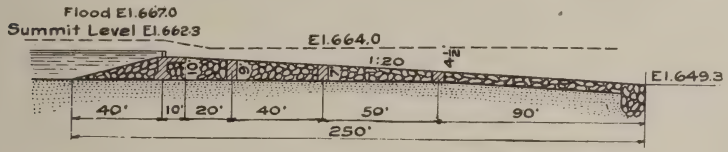

Fig. 25. Okhla weir across the Jumna River, India.

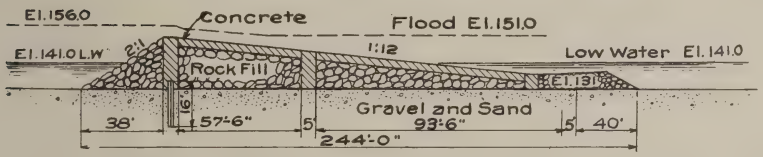

Fig. 26. Laguna weir across the Colorado River, U. S. A.

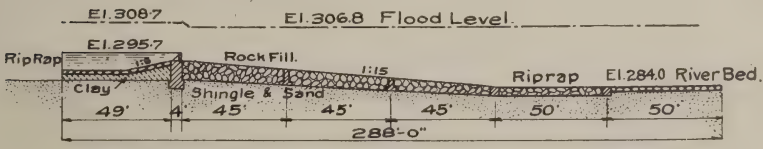

Fig. 27. Madaya weir across the Madaya River, Burmah.

passed under the main body of the dam and lifted this 100 feet of fore-apron, as was naturally to be expected.

Comment: Such proceeding was so natural that it is not mentioned in the preceding discussion of upward water pressure in this paper. As the remaining dam had a percolation factor of only 2.4, the underlying material naturally blew out, and the dam failed. Mr. Bligh's percolation factor for coarse sand is given as 12, and for boulders and gravel as 4 to 6. As the fore-apron lifted so completely, there is no data in this failure to indicate whether 6 is or is not a proper percolation factor for the material in this case, which was composed of boulders, gravel and sand.

71. Experiments were made by Major Jervey at Wheeling to determine percolation factor for boils and blow-out. A few of the results are given below. The percolation factors obtained are very much smaller than was anticipated before the experiments were started.

72. Experiment—

Material: Alluvial soil, Fig. 11.

Object: To determine percolation factor for quick blow-out.

Conditions: The soil was placed to the elevation of the bottom of the slab, and as the experiment progressed layers were removed from the "downstream" side of the sheet piles and later the thickness on the "upstream" side was decreased. No slab.

Results: Practically no warning by boils; blow-out with percolation factor of about 0.08.

Head.	Time standing.	Length of travel for percolation.
5 ft. 6 in.	40 min.	48 in.
6 ft. 0 in.	12 hrs.	42 in.
6 ft. 6 in.	2 hrs.	36 in.
6 ft. 9 in.	2 hrs.	33 in.
7 ft. 0 in.	1 hr. 40 min.	30 in.
7 ft. 3 in.	42 hrs.	27 in.
7 ft. 11 in.	1 hr. 10 min.	22 in.
7 ft. 11 in.	1 hr. 15 min.	16 in.
7 ft. 11 in.	2 min.	9 in. blow-out.

73. Experiment—

Material: Sand, Fig. 13.

Object: To determine percolation factor for boils and blow-out.

Conditions: Sand was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet.

Results: Boils appeared at 5 feet of head, factor of .40; quick blow-out came at 6 feet of head, being a percolation factor of 0.33.

Head.	Head held.	Notes.
Feet.	*Hours.*	
1	4	Water turned on 6.00 a. m., April 24, 1914.
	8	
	24	No blow-out.
2	4	Water turned on 7.00 a. m., May 12, 1914.
	8	
	24	
3	4	Water turned on 7.00 a. m., May 13, 1914.
	8	
	24	
4	4	Water turned on 7.00 a. m., May 14, 1914.
	8	
	24	
5	4	Water turned on 7.00 a. m., May 15, 1914.
	8	1 very large boil, edge of slab.
	24	Do.
6	3	Water turned on 7.00 a. m., May 21, 1914. 1 very large boil, edge of slab. Blow-out 10.00 a. m.

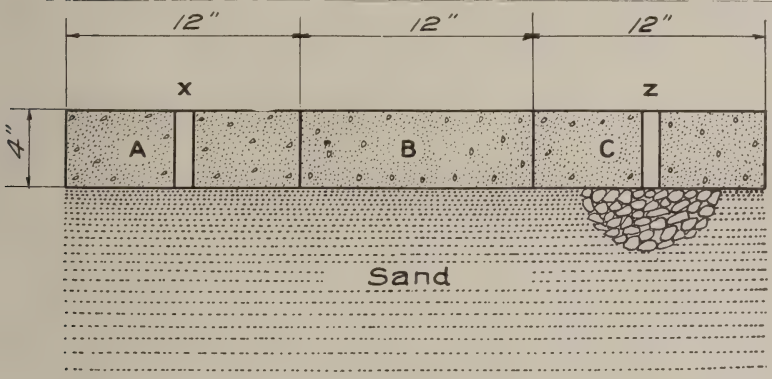

Fig. 28.

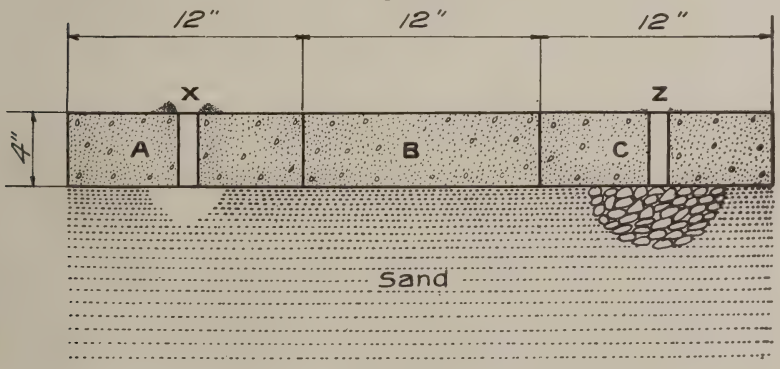

Fig. 29.

74. Experiment—

Material: Gravel aggregate, Fig. 13.

Object: To determine percolation factor for boils and blow-out.

Conditions: Gravel aggregate was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet.

Results: Boils appeared at 1 foot of head, factor of 2.0; quick blow-out came at 5 feet 9 inches of head, being a percolation of 0.35.

Head.	Head held.	Notes.
Feet.	*Hours.*	
1	4	Water turned on 8.30 a. m., April 1, 1914. Two small boils 1 inch from slab.
	8	
	24	
2	1	Two small boils carrying muddy water.
	5	Six small boils carrying muddy water.
	22	Six small boils carrying muddy water; (no blow-out).
3	3	Six small boils 6 inches from slab, and 2 large boils 3 inches from slab.
	8	Six small boils carrying clear water. Large boils had disappeared.
4	4	Water turned on 9.00 a. m., April 3, 1914. Four small boils and one large one 1 inch from slab. Water supply was not sufficient for more than four hours.
5	4	Water turned on 8.45 a. m., April 9, 1914. Six small and two large boils 10 inches from slab. Water supply was not sufficient for more than four hours.
5¾	½	Water turned on 12.30 p. m., April 13, 1914. Seven small and three large boils running muddy water 15 inches from slab. Blow-out 1.00 p. m. Head, 5 feet 9 inches.

75. Experiment—

Material: Washed gravel, Fig. 13.

Object: To determine percolation factor for boils and blow-out.

Conditions: Washed gravel was placed as shown in Fig. 13, thus giving a least length of travel of over 2 feet; inspection made and results noted at times indicated.

Results: Water ran through as fast as pipe could supply it. No blow-out.

76. EXPERIMENT—

Material: Sand, Fig. 11.

Object: To determine if sand could be blown out through holes in the slabs; question of quick blow-out.

Conditions: Slabs A, B, and C in place; a bulkhead 3 feet out from the sheet piles was used. Slabs A and C each had a hole 1 inch in diameter in the center, referred to as "X" and "Z." "Z" had an inverted filter about 7 inches in diameter and 3 inches deep built up of graded gravel inserted in the sand underneath, while the sand came directly to the concrete under "X," as shown in Fig. 28.

Results: A small amount of sand was carried out of the holes and formed a low crater around the edge (see Fig. 29); after which the sand merely boiled in the hole and crater. No "blow-out" occurred. (Exactly this same result has been found in a lock floor now actually in operation at Dam No. 13, Ohio River.)

With "X" open and "Z" closed, the results were as follows:

Head.	Head held.	Tubes.			Notes.
		A	B	C	
Inches.		*In.*	*In.*	*In.*	
20	0	1½	1½	1½	
20	10 min.	1½	1½	1½	Sand moving in "X."
30	0 rising	1½	1½	1½	Slight ridge of sand formed crater around hole. Boiling in hole.
48	0 rising	1⅞	1⅞	1⅞	Crater slightly increased in height. Violent boiling in hole.
64	10 min.	2	2	2	Violent boiling in crater.
64	10 min.	2	2	2	Boiling of larger sand in crater, but no sand carried away from hole.
With "Z" open and "X" closed, the results were as follows:					
44½	0 rising	3	2½	1½	Water flowing clear.
64	10 min.	3¼	2¾	1½	Do.

77. EXPERIMENT—

Material: Sand and gravel, Fig. 11.

Object: To determine action of a layer of gravel on sand with regard to preventing blow-out under sheet piles; question of quick blow-out.

Conditions: No slab was placed. Sand was placed in the stack to an elevation 15 inches above the bottom of the sheet piles, and on the "downstream" side of the sheet piles sand was placed to the elevation of the bottom of the sheet piles. A bulkhead 3 feet out

from the sheet piles was used. Gravel was placed between the bulkhead and the sheet piles to an elevation 26 inches above bottom of sheet piles.

Results: No sand was carried out through the gravel, and quick blow-out occurred only after all the gravel was removed; that is, when the length of travel was only 15 inches.

Head.	*Head held.*
28 inches.	0 rising.
68 inches.	0 ——
68 inches.	14 min.
80 inches.	0 falling.
78 inches.	__; no sand was carried out.

A layer of gravel 1 foot thick was removed.

Head.	*Head held.*
80 inches.	0 ——
80 inches.	8 min.; no sand was carried out.

A layer of gravel 6 inches thick was removed, leaving 8 inches of gravel in place.

Head.	*Head held.*
80 inches.	0 ——
80 inches.	4 min.
80 inches.	0 rising; no sand was carried out.

A layer of gravel 4 inches thick was removed, leaving 4 inches of gravel in place.

Head.	*Head held.*
86 inches.	5 min.; no sand was carried out.

The remaining gravel was removed, and blow-out occurred at once.

78. Experiment—

Material: Sand and gravel, Fig. 11.

Object: To determine if a layer of gravel on sand will prevent sand being carried through relief holes in the slab; question of quick blow-out.

Conditions: Sand was placed in the stack to an elevation 15 inches above the bottom of the sheet piles; and on the "downstream" side of the sheet piles sand was placed to the elevation of the bottom of the sheet piles. A bulkhead 3 feet out from the sheet piles was used. Gravel was placed between the sheet piles and the bulkhead to the elevation of the bottom of slabs. Three slabs were placed, slabs "A" and "C" each having a hole 1 inch in diameter in the center, referred to as "X" and "Z."

Results: With heads of from 2 to 4 feet shown in the tubes, holes "X" and "Z" were opened separately or together. The water

flowed out quickly, head fell in tubes to 1 to 3 inches, but no sand was carried out from "X" and "Z," thus showing that the layer of gravel prevented the carrying out of sand through the vent holes under pressure.

79. EXPERIMENT—

Material: Sand and gravel, Fig. 11.

Object: To determine a percolation factor for quick blow-out under sheet piles, and the relative effect on this percolation factor of a layer of sand and a layer of gravel on the outlet surface; question of quick blow-out.

Conditions: A bulkhead 3 feet out from the sheet piles was used, but no slab was placed. The head of water was slowly increased until quick blow-out was obtained.

(*a*) Sand was placed on both sides of the sheet piles to an elevation 6 inches above the bottom of the sheet piles.

(*b*) The sand was replaced as above, and a 6-inch layer of gravel was placed on the sand between the bulkhead and the sheet piles.

(*c*) The sand was replaced as in (*a*), and another 6-inch layer of sand was placed on this sand between the bulkhead and the sheet piles [in place of the gravel described in (*b*)].

Results: Percolation factor for quick blow-out: (*a*) .21; (*c*) .26. The 6-inch layer of gravel (*b*) delayed the blow-out longer than the 6-inch layer of sand. The gravel, in addition to its porosity, was slightly heavier than the sand, but no special relation of weight to delay in blow-out was noticed.

Head.	Head held.	Notes.
Inches.		
(*a*) 19	0 rising	Started to boil.
56	----------	Blew out.
(*b*) 50	0 rising	
61	0 falling	
54	0 rising	
70	0 falling	
65	0 rising	No sand appeared yet.
78	----------	Blew out.
(*c*) 44	0 rising	
49	0 rising	Started to boil.
69	----------	Blew out.

80. EXPERIMENT—

Material: Gravel aggregate, Fig. 11.

Object: To determine the head necessary for blow-out of gravel aggregate through the relief holes in concrete slabs.

Conditions: A bulkhead was used 3 feet out from the sheet piles. The aggregate was placed to the elevation of the bottom of the slab on both sides of the sheet piles, and three concrete slabs were placed. Slabs *A* and *C* each had a hole 1 inch in diameter in the center, referred to as "X" and "Z."

Results: A small amount of the fine dirt was carried out of the relief holes, but no blow-out came with the relatively large heads. With "X" open and "Z" closed, the results were as follows:

Head.	Head held.	Tubes.			Notes.
		A	B	C	
Inches		*In.*	*In.*	*In.*	
17	0 rising	2	2	2	Carrying out dirt.
22	Do.	2¾	2¾	2¾	Do.
28	Do.	3	3¼	3½	Do.
34	Do.	4	4½	5	Do.
40	Do.	5	5½	6	Do.
46	Do.	6	6⅜	7	Do.
52	Do.	7	7⅜	8	Do.
58	Do.	8½	9¼	9½	Do.
64	Do.	9½	10¼	10¾	Do.

Dirt only, no sand, was carried out through "X."

With holes "X" and "Z" open, the results were as follows:

Head.	Head held.	Tubes.			Notes.
		A	B	C	
Inches.		*In.*	*In.*	*In.*	
64	0 falling	3¼	2½	1½	Carrying out dirt
58	Do.	2½	2	1½	Do.
52	Do.	2	1½	1	Do.
46	Do.	1⅞	1⅜	1	Do.
40	Do.	1¾	1¼	1	Water flowing clear
34	Do.	1½	1	¾	Do.
28	Do.	1⅛	⅞	⅝	Do.
22	Do.	⅞	¾	½	Do.
17	Do.	⅞	⅝	½	Do.

Dirt carried out, but no sand.

81. EXPERIMENT—

Material: Washed gravel, sand in holes, Fig. 11.

Object: To determine the effect on blow-out of introducing sand through holes in a concrete slab without drain, and ramming this sand into the gravel; question of quick blow-out.

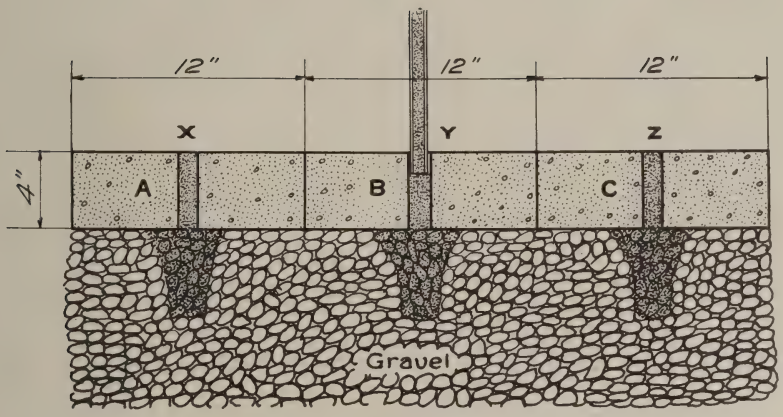

Fig. 30.

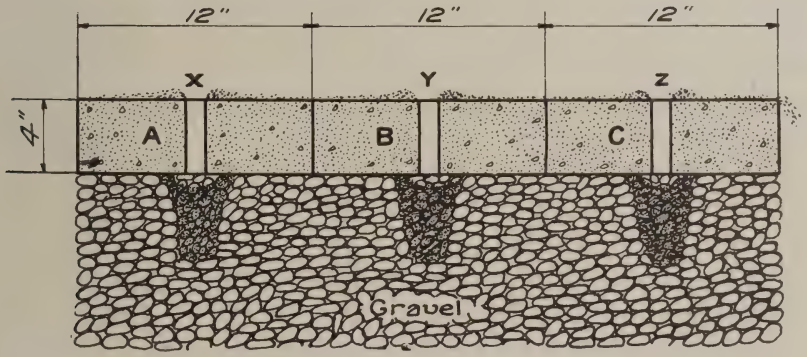

Fig. 31.

Conditions: A bulkhead was used 3 feet out from the sheet piles. Gravel was placed to the elevation of the bottom of the slab. Three slabs were placed, *A*, *B*, and *C*, each having a hole 1 inch in diameter in the center, hereafter referred to as "X," "Y," and "Z." Sand was rammed through "X" and "Z" into the gravel for a depth of about 6 inches, and, as was found on removing the slab, spread about 1 inch each side of the center of the hole. A ¾-inch pipe was calked into "Y," the top of this pipe being 7¼ inches above the top of the concrete. Sand was rammed into the gravel through this pipe as in "X" and "Z," after which the pipe itself was packed with sand. (See Fig. 30.)

Results:

Head.	Remarks.
Inches.	
18½	Sand in hole "X" started to boil.
24	Water flowed from top of pipe in "Y."
54	Sand "blew out" of hole in "Z."

The sand in "X" had not blown out at 54-inch head, so it was disturbed with a rod to a depth of about 2 inches, when it suddenly blew out. Water had been slowly percolating through the pipe in "Y;" the pipe was pulled out, and the sand was immediately blown out of the hole in the slab.

When the slabs were removed, it was found that the sand was washed away immediately under the slab at "X," "Y," and "Z," but was still in place at the depths greater than 1 inch. (See Fig. 31.)

SUMMARY OF CONCLUSIONS.

82. Little or no allowance can safely be made for adhesion of concrete to concrete or of concrete to rock.

83. Water pressure is transmitted through concrete, though quite slowly. If allowed free exit, the pressure is practically negligible.

84. Water pressure is transmitted through joints between concrete and concrete, and is transmitted more freely than through solid concrete. The water is transmitted quite slowly; and, if allowed free exit, the pressure is practically negligible.

85. Water pressure is transmitted through joints between concrete and rock. This is generally transmitted more freely than through joints between concrete and concrete. The amount varies

with different rocks. The water is transmitted quite slowly, in general, and, if allowed free exit, the pressure is practically negligible in good rock without cracks or fissures.

86. Water travels in small veins in the joints between concrete and concrete, and between concrete and rock. If allowed free exit, the pressure varies irregularly between upper and lower pools; if exit is closed, the pressure quickly becomes that of upper pool, and water rises in test holes to level of upper pool as soon as enough water has passed through the veins to fill the test holes.

87. The amount of space of these small veins; that is, the area of upward water pressure, varies from nearly zero in excellent granite foundations to 50 per cent or more in rotten shale. It is practically impossible to calculate this.

88. Water pressure is transmitted more or less freely through washed sand, gravel aggregate, and washed river gravel; and hardly at all through alluvial soil.

89. Puddled alluvial soil has practically no water pressure, if there are any drains; but such soil quickly disintegrates with a running leak.

With a material such as alluvial soil, which offers so great resistance to percolation as to practically prevent it, a very small number of relief holes in the slab would be sufficient to practically eliminate the upward water pressure, provided, of course, that the material were such that only percolation and no erosion took place.

90. With a material offering so great a resistance to percolation as washed sand, a very small number of relief holes will greatly reduce the upward water pressure when there is no outlet at the toe of the slab.

With a slab having free exit for water at the toe, and a row of sheet piles at the heel, the upward water pressure varies from one-third the head at the heel to one-fourth the head at the toe.

91. Gravel aggregate offers less resistance to percolation than the river sand, and more than the river gravel. Hence, for the same reduction of upward water pressure when there is no outlet at the toe of the slab, more relief holes will be needed than for the sand and less than for the washed gravel.

For a slab having free exit for water at the toe and a row of sheet piles at the heel, the upward water pressure varies from one-sixteenth the head at the heel to one-fortieth the head at the toe.

92. In a material offering such a small resistance to percola-

tion as washed gravel a large number of relief holes in the slab will be necessary to materially reduce the upward water pressure when there is no outlet at the toe of the slab.

With an outlet for water at the toe of the slab and a row of sheet piles at the heel, the upward water pressure varies from full head at the heel to about one-third the head at the toe.

93. The pressure of water with free exit, transmitted through sand, washed gravel, or gravel aggregate, is greatly diminished on entrance and more slowly, but surely, diminished along line of travel.

94. Full upward pressure of water, without free exit, is transmitted quickly through washed gravel, and less quickly through gravel aggregate and sand.

95. Water percolates (very slowly) through concrete and rock.

96. There is very little percolation through alluvial soil.

97. Water percolates more or less freely through sand, gravel aggregate, and washed gravel.

98. Water creates "boils" at the toe of the foundation, after the relation of head to length of travel (percolation factor) becomes a certain amount, varying for different materials. This condition is dangerous to the foundation when the velocity of exit is sufficient to carry away with it the material of the foundation. It is generally best to make this percolation factor so great that there will be no such boils.

99. If the percolation factor is too small (varies for different materials), the material under the foundation will blow out and the foundation will fall in. With alluvial soil, this blow-out comes with little or no previous warning by boils.

DESIGN.

100. In considering the subjects of Adhesion, Percolation, and Upward Water Pressure, before studying past experiments and before experiments were made by the Ohio River Board, it was thought that the design of dams must be based to a great extent on guess work (in fact, that seems to be the general method of design at present, the dam being made unnecessarily safe if the money is available); but experiments show that there are certain fixed values which may be determined by past or future experiments, although the design can not yet be based absolutely on scientific deductions.

From the preceding conclusions, it is my opinion that experiments should be made at practically every site, as was done at

Panama, to determine the upward water pressure and the factors for percolation.

However, in case no such experiments can be made, the results of experiments herewith and other experiments can be used to decide upon fairly accurate assumptions, due regard being paid to similarity of material.

It is specially to be noted that key, or sheet piles, at the toe of a dam are conducive to increase of upward pressure and consequently should be omitted at all times.

101. Before deciding on the design for any dam, consideration must be given, as far as applicable, to failure in the following ways:

Sliding;	Crushing;
Overturning;	Blow-out.

Each of these is considered below and suitable types of dams are shown to eliminate these difficulties.

Sliding.

102. If founded on piles, it is customary and natural to place the tops of the piles some 18 inches into the concrete, so as to prevent sliding. Calculations are easily made, with the values in hand-books, to determine the shearing values of the piles. If the material is very soft, experiments should be made to determine the lateral bearing value of each pile; if not very soft, the lateral bearing value of each pile is sufficient and can be determined from the hand-books. See Fig. 32, showing the piles for fixed weir at Dam No. 43, Ohio River, where the material is sand, very fine but well packed.

103. If the dam is founded on rock, it is claimed by many engineers that the adhesion of concrete to the rock will be sufficient, or at least of great value. Experiments do not prove this. Those made by Colonel Poe (see paragraph 9) are strictly laboratory experiments. The experiments to show adhesion of concrete to concrete made by Professor Johnson at Harvard and Mr. Robinson in England give remarkably high values in a laboratory (see paragraphs 7 and 8); but those made by Colonel Shunk at Pittsburgh (see paragraphs 10 and 11) show that these values are entirely unreliable in actual construction. The Ohio River Board has decided to place no reliance on adhesion of concrete to concrete or to rock, and has fixed upon a key in all cases to prevent sliding. The depth of this key is placed at "never less than 3 feet" in

order to prevent possible shearing off of corners. See Fig. 33, showing design for fixed weir at Dam No. 39, Ohio River.

This key acts also as a cut-off wall and is consequently placed near the heel. If intended solely to prevent sliding, there would be no particular objection to placing it near the toe as advocated in *Engineering and Contracting* (April 1, 1914, page 376), provided it has drains through it to prevent accumulation of upward water pressure.

Overturning.

104. That there is always upward water pressure on sand and gravel foundations is evident from the experiments by Mr. Saville at Panama (see paragraph 27) and from the experiments by Major Jervey at Wheeling (see paragraphs 33-46). The experiments of Major Jervey also show that this pressure becomes equal to the full head of upper pool if there is no drainage into lower pool; but if there is such drainage, the pressure decreases from upper pool to lower pool, though it is not proven that the pressure decreases uniformly from full head of upper pool at heel to full head of lower pool at toe. It seems more probable that some head is lost on entrance into the material, and some on discharge from the material.

With such foundations as sand and gravel, the Ohio River Board has decided to provide drainage holes in sheet piles on lower edge of the concrete (or have no sheet piles at all); and to make design to withstand an upward water pressure over whole base of the dam varying in force from head due to upper pool at upstream edge to head due to lower pool at downstream edge. See Fig. 32, showing drain holes in sheeting on downstream side of design of fixed weir at Dam No. 43, Ohio River, where material was such as to render sheeting necessary.

105. With foundations on rock, the amount of upward water pressure is not so easily determined. It is evident, however, that there is such pressure.

Experiments by Mr. Francis (see paragraphs 16 and 56), by Colonel Hains (see paragraph 57), Colonel Shunk and Major Jervey show that water percolates through concrete and through joints between concrete and concrete or rock.

Experiments as reported by Mr. Weidner (see paragraph 27), by test holes at Gatun Locks (see paragraph 28) and by Colonel Shunk and Major Jervey show that the water will rise in any test hole, and that roughly the final height of such rise varies about as

its distance from lower pool relative to the distance between pools. These experiments show also that in some cases the water reaches final height in the hole very quickly, while in others it takes a very long time, thereby indicating that in general the water is present in very thin streams; and if efficient drains are introduced the upward water pressure beyond such efficient drains will be practically negligible in good rock. In this connection, it is well to note that the drains must be effective (must not be too small and must not be stopped). The floor of the Lost River Dam was lifted by upward water pressure because half of the weep holes were stopped (see *Engineering News*, April 30, 1914, page 968).

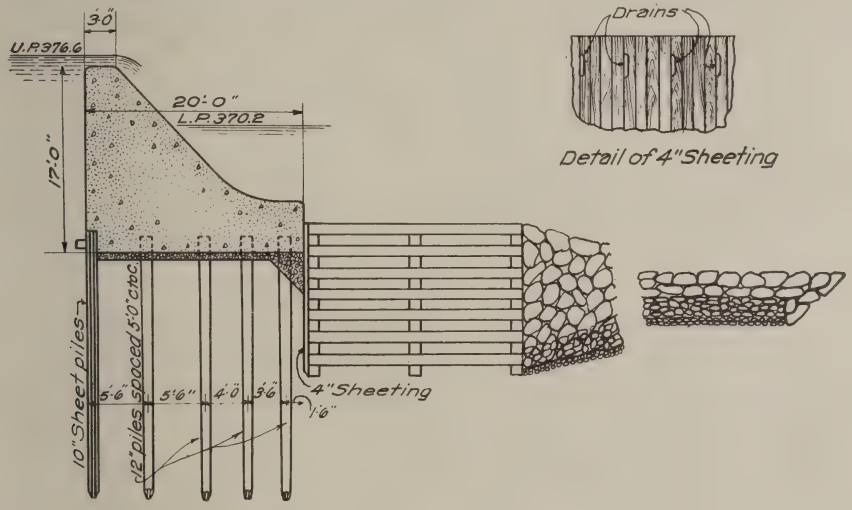

Fig. 32.

With rock foundations, the Ohio River Board in the general case uses the following values:

(*a*) Upward water pressure varying from upper pool at upstream edge to lower pool at drains.

(*b*) Pressure under 50 per cent of base for firm solid rock and under 100 per cent of base for weak rock.

(*c*) Drains in rock below key (see Fig. 33).

(*d*) Neglecting upward water pressure, center of pressure to fall within middle third; including upward water pressure, to fall far enough inside the toe so that the maximum compression shall not exceed 400 pounds per square inch for good rock or 70 pounds per square inch for poor or doubtful rock.

To overcome upward water pressure gravity sections are gen-

erally used, but sometimes steel rods with or without initial tension are used to anchor foundations to rock.

106. The system so often used of building a key, which is called a "cut-off wall," at or near the upper edge of a foundation is all right in theory. That of building also a "cut-off wall" at the lower edge of the wall is about the worst thing that could be done. The upper key cuts off a little water, although its principal value lies in the fact that the water can drain away below it faster than it can pass along the more difficult joint of the key itself. But if a second key is placed at the lower edge of the foundation and there are no drain holes through this key, this water accumulates under the center of the foundation, with much resultant upward pressure.

The key should not be too close to the upper edge of the foundation, as too free passage of water will create damage in poor rock; and real drains should be provided below it to the lower pool to provide better drainage than now provided by poor workmanship, which is useful in that it leaves a kind of drain along the joint between the concrete and the rock.

Crushing.

107. Dams fail by crushing of the rock foundation or by bearing down or rupture of the pile foundation. The force of this pressure depends upon all of the forces acting, which, of course, requires a determination of the value of the upward water pressure. This upward water pressure is particularly important in that it changes position of the center of pressure, generally moving it nearer the toe and, consequently, increasing the unit pressure at and near the toe.

108. Having determined the maximum unit presure which is possible, the material must be examined or tested to determine its strength. If rock, laboratory tests are most satisfactory; but results of experiments elsewhere would be somewhat dependable. If sand, gravel, etc., a pile can be driven and weighted, or results of experiments elsewhere with similar material would be somewhat dependable.

Blow-out.

109. When the water percolates through a dam with considerable speed and runs out on downstream side with considerable velocity (pressure), there is danger that the water will carry with it certain material lining the vein of its passage, thus increasing the

size of the vein. This is progressive; the size of the vein continues to increase until a vast quantity of water rushes through, the vein becomes an enormous hole and the concrete monoliths above collapse from lack of support. This is what happened to the Stoney River Dam and to many dams in India.

110. That the percolation factor is too small is first seen by the appearance of "boils" on the downstream side. These are sometimes present for days in sand and earth dams; but if there is much alluvial soil present, there is short shift between the boil and the blow-out.

On the other hand, if the boil does not carry material (is not muddy), it is probable that it is formed by the junction, near exit, of several veins through the dam, and destruction will not follow.

Quite often it is possible to build a wall around these boils,

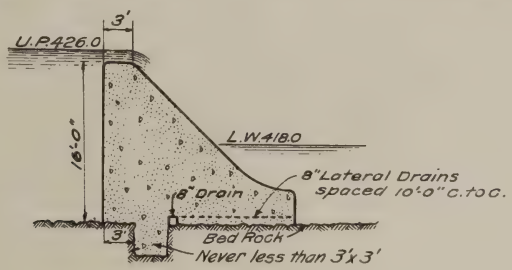

Fig. 33.

letting the water rise in this pool until the difference of pressure is so decreased that the velocity is not sufficient to carry away material and enlarge the veins.

Although, as stated, boils do not always cause destruction, their presence is an indication that prompt action is necessary.

111. There are no very definite experiments as to percolation factors needed to prevent blow-out. Any number of experiments have been made in connection with filtration to determine amount of percolation through sand and other materials. The amount of such percolation is not of great value, except to determine the size of drains needed to carry away percolated water in order to prevent accumulation of head.

112. Percolation factors for safety against blow-out have been assumed by Mr. Bligh (see paragraphs 68-70); but he does not say on what they were based. Presumably, they were based on the fact that smaller factors were present in dams that failed; but this

is not entirely conclusive, for in some cases it was evident that other causes contributed to the failure of the dams.

113. Experiment by Major Jervey (paragraph 76) shows that sand will blow out through a drain hole until the exposed surface of the sand is sufficiently large so that the velocity of water is not great enough to move the sand. Such first movement of sand is not a blow-out, and if gravel is first placed so that it will fill the space of such a crater, no sand will be blown out. Although it is evident that such a construction is safe, there is not sufficient data yet to determine exactly what is the percolation factor for safety.

114. For *earth dams,* the most suitable design is shown in Fig. 34, this being an Ambursen type. This is particularly suitable to eliminate blow-outs and will not fail if the following conditions are fulfilled:

(*a*) If there are no large holes or cracks leading from saturated

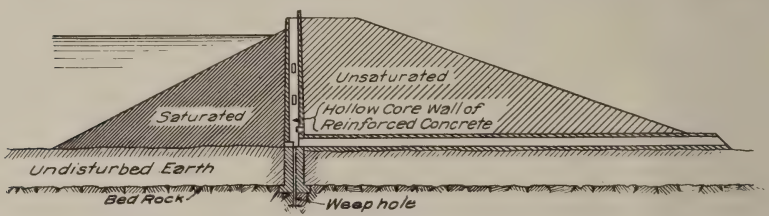

Fig. 34. Section of earth fill dam with hollow core walls.

earth to inside of core wall, or from unsaturated earth to lateral drains.

(*b*) If length and resistance of line of travel through saturated earth, down around and under core wall (whether on rock or earth), through unsaturated earth, is such that the percolation factor is a safe one—that is, such that there will be no carrying of material and enlargement of veins.

(*c*) If the weight back of wall is sufficient to prevent displacement of the wall.

(*d*) If there is no overflow to wash away the rear unsaturated earth.

115. For rock foundations, a suitable design for safety against blow-out is shown in Fig. 33. In such dams it is only necessary to remember that rotten rock, hardpan, etc., can be carried away by rapid and continual percolation, and the beginnings of such drains must be far enough from the upper pool to prevent destructive percolation. The Ohio River Board has adopted a tentative dis-

tance of 5 feet for ordinary shale, but this distance will vary with the material and the depth of upper pool.

If the drain holes are placed too far from the upper pool, the upward water pressure will be considerably increased, and the dam will cost more.

Further Experiments.

116. There is enough data now available from past experience or past experiments to make fairly accurate calculation in order to insure safety against sliding, crushing, and blow-out with foundations on rock, sand, gravel, etc. There is a great deal of information as to upward pressure with foundations on sand, gravel, etc.; but there is practically no information available as to upward pressure with foundations on rock. It is known that there is such pressure; but it can not be calculated from the rate of rise in test holes. Until accurate experiments are made to determine the exact force of this upward water pressure with different rocks, assertions may be confidently made that it is 3 per cent with shale or 96 per cent with granite, and there will be no proof to the contrary.

The Selection and Occupation of Lines of Battle.*

BY

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"A military position is any site which is or may be occupied by troops for the purpose of receiving or making an attack." This definition includes both those localities that are deliberately selected prior to their occupation and those that result from the successive movements of the several fractions of an army in the attack.

An army on the offensive moves to the attack as a whole, but not often are all of its parts equally successful; for the fractions work forward as the result of local successes, which rarely advance the whole line simultaneously. The places that are reached by the successful units are held if possible. The men utilize the natural strength of their new situations and improve them with trenches, which thus establish the new position of the army. Not these positions, but those that are occupied by the army on the defensive are here discussed.

Although offensive fighting only gives victory, yet it may sometimes be expedient for an army to fight defensively; but the assumption of the defensive usually implies a temporary weakness on the part of the defender, or a desire to delay the final issue of battle, until his preparation is complete and his army thoroughly mobilized and equipped, and ready to take the field against the enemy with the chances of success in his favor. The defensive attitude must not become habitual, nor should it be maintained longer than the day that preparations for an offensive campaign are complete.

Passive defense saps the strength of an army as does defeat. The spirit that wins is a fierce desire to attack and beat the enemy; but desire alone is not enough, there must be action.

Strategy selects, tactics occupies, and fortification strengthens

*This article was written before the beginning of the present European War. It is of especial interest at this time.

the position. Strategy is the province of higher commanders; tactics and fortifications are the provinces of subalterns. The position may be selected for various purposes. It may be taken up to defend a political or a commercial center or a strategic locality; to delay the enemy or to compel him to take another route or position; or to permit the maneuvering of the defender's army; or it may be taken to invite an assault on a selected battlefield.

A position may be determined and limited by well-defined natural features; such as a range of hills, a divide between valleys crossing the line of operations, or it may be generally parallel to a stream course. Only in featureless country is its proper location poorly defined. Even then, the location of the roads, villages, woods, etc., will influence its choice.

In selecting the position, strategy chiefly considers the following requisites:

(*a*) A location that will compel the enemy either to attack or to abandon his advance, and, at the same time, one that will be favorable to the assumption of the offensive;
(*b*) An extent appropriate to the strength of the command;
(*c*) Flanks that can be well protected.

Bearing in mind that a position has a defensive value only when it compels the enemy to attack it, or when his attempts to evade such an attack bring about conditions favorable to the defense, it follows that the line of resistance must be interposed between the defended area and the enemy. The question of the most probable point of attack does not enter. Every approach must be defended and the line must completely guard the protected area. The line of defense must then be virtually continuous.

The general location of this line of resistance is largely determined by the range of weapons that will be employed by the enemy. The effective range of heavy artillery is, say, 6,000 yards. The line of defense then, must prevent the enemy from occupying positions at a distance less than, say, 3 miles from the limits of the protected area. It must be so situated that the enemy can not operate against the defended area while the line is intact.

The consideration of economy of troops and material for defense and of labor and material and time for the construction of the entrenchments, imposes the further condition that the line be drawn in as much as possible, so that its development be not excessive. The determination of its approximate situation can best be made by a painstaking study of accurate maps, upon which are shown

relative directions and distances between positions in the area studied, as well as differences of elevation that affect command, and with which the strategical use to be made of prominent features, such as rivers, lakes, swamps, villages, woods, etc., can be fully considered.

The final selection of positions, however, can by no means be made in an office. It is essential that the choice of lines of trenches, the selection of sites and traces of redoubts and the determination of artillery positions be made in the field.

In estimating the front that can be held against an ordinary frontal attack, the calculations are based on the amount of infantry available, and may be roughly made as follows: "For a brigade of 4,500 men in the fighting line one-third may be assigned to the local reserves, one regiment of the brigade or one battalion of each of the three regiments. Six battalions or twenty-four companies will then be available for the fighting line and supports. Each battalion or company furnishes its own support, ordinarily about one-fourth of its strength. The total front covered by the brigade will be about 4,500 yards, or 1 yard per man." Thus it is seen that the strength of the infantry largely determines the front that should be occupied.*

Frontal assaults are readily resisted. Prepared military positions are usually carried after prolonged attacks and sieges only, when the positions have perhaps already served their purpose, or they are carried by operations against their flanks. Recent wars accentuate the lesson learned from our Civil War: that it is exceptional when frontal attacks succeed. The position must be taken by flank movements or by operations against the line of communications. Especial care must, therefore, be given to the security of the flanks, and all possible means should be provided against being outflanked or maneuvered out of position.

The other considerations that influence the selection of the position, the most important of which are noted below, will be briefly discussed later. They are: the force to be resisted; the time available for strengthening the position; the troops and civilian labor that can be used on the work; and the duration of the defensive rôle and the plans for the subsequent assumption of the offensive.

Our Field Service Regulations state that a reconnoissance of the

*These figures are taken from "Field Service Regulations;" it must be understood, however, that they are variable. No fixed rules can be given.

position should be made by the commander of the troops assigned thereto. He should be accompanied by officers of the General Staff, Artillery and Engineers. The purpose of this reconnoissance is to determine the best method of occupying the position and to formulate a "plan of defense" to be issued as a "defense order."

In the Russo-Japanese War, General Kuropatkin commanded the Russian forces. At the beginning of the war, after a study of the theater of operations, he ordered the fortification of the city of Liao Yang, as a strategic center for maneuvering the army. Months afterwards, when in the progress of the war its occupation became necessary, his plan of defense was made and his defense order was published, which is as follows (see map):

LIAOYANG, *August 16 (29), 1.15 a. m.*

On the 15th (28th) of August, the first, second, and third armies of the enemy have occupied a front from Peikou, valley of the Tangho, to Wangpaotai, Changchiaputzu, Shako, and Tuntai.

My army will assemble in the district of Liaoyang. The extreme right flank of the army will be guarded by the detachment of Major General Kosagovski (2 battalions, 3 sotnias, and 6 guns at Tawan and by 2 companies, 2 guns, and a half sotnia at Hsiaopeiho).

The extreme left flank will be protected on the line Pensihu-Taling-Hsingking by 4 battalions, 8 guns, and 6 sotnias.

In case of a further advance of the enemy the army will fight in the fortified position Mayetun, Tatzuying, Hsiaoyangtzu, Hsinlitun, Wijakou, Kuchiatzu, Yayuchi, Hsiaopu, Sanwantzu, Tatzupu, Hsikuantun.

I. FIGHTING GROUP.

a. Lieutenant General Baron Stackelberg, First Siberian Corps, First East Siberian Rifles Division, First East Siberian Rifles Artillery Brigade, Ninth East Siberian Rifles Division, Ninth East Siberian Rifles Artillery Brigade, Ussuri Cavalry Brigade, First East Siberian Sapper Battalion (24 battalions, 64 guns, 6 horse guns, 8 machine guns, 10 sotnias of cavalry, 1 sapper battalion), will defend the position on the heights north of Mayetun, Tatzuying, Hsiaoyungtzu, Hsinlitun, guard the grounds on the left as far as the creek flowing from Sanchiatzu to Tassu, and maintain communication with the Third Siberian Corps.

b. Lieutenant General Ivanov, Third Siberian Corps, Third East Siberian Rifles Division, Third East Siberian Rifles Artillery Brigade, Sixth East Siberian Rifles Division, Sixth East Siberian Rifles Artillery Brigade, First and Sixth Horse Mountain Batteries, Second Chitinski (Cossack) Cavalry, Second East Siberian Sapper Battalion (24 battalions, 64 guns, 8 horse mountain guns, 6

sotnias, 1 sapper battalion), will defend the heights north of Kuchiatzu as far as the Suichangyu-Mengchiafang road, will guard the ground on the right as far as the creek from Sanchiatzu to Tassu, and will maintain communication with the First Siberian Corps.

c. Lieutenant General Sluchevski, Tenth Army Corps, Ninth Infantry Division, Ninth Artillery Brigade, Thirty-first Infantry Division, Thirty-first Artillery Brigade, First East Siberian Mountain Battery, First and Second East Siberian Mortar Batteries, First Orenburg Cossack Regiment, Sixth Sapper Battalion (32 battalions, 112 guns, 8 mountain guns, 12 mortars, 6 sotnias, 1 sapper battalion), will defend the line from the Suichangyu-Mengchiafang road to Hsiaopu.

d. General of Cavalry Baron Bilderling, Seventeenth Army Corps, Third Infantry Division, Third Artillery Brigade, Thirty-fifth Infantry Regiment, Thirty-fifth Artillery Brigade, Two hundred and eighty-second Infantry Regiment, Sixth Battery Twenty-eighth Artillery Brigade, Second Independent Cavalry Brigade (Fifty-first and Fifty-second Dragoons), Ussuri Cossack Regiment, Terek-Kuban Cossack Regiment, Fourth Trans-Baikal Cossack Horse Artillery Battalion, Seventeenth Sapper Battalion (36 battalions, 120 guns, 6 horse guns, 23 sotnias, 1 sapper battalion), will cover the left flank and defend the position on the right bank of Taitzu, will observe the river above Hsikuantun, and maintain communication with the detachment at Pensihu.

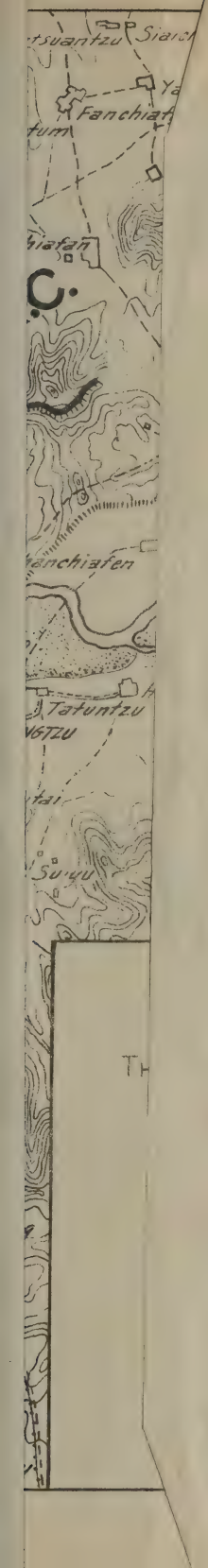
II. GENERAL RESERVE.

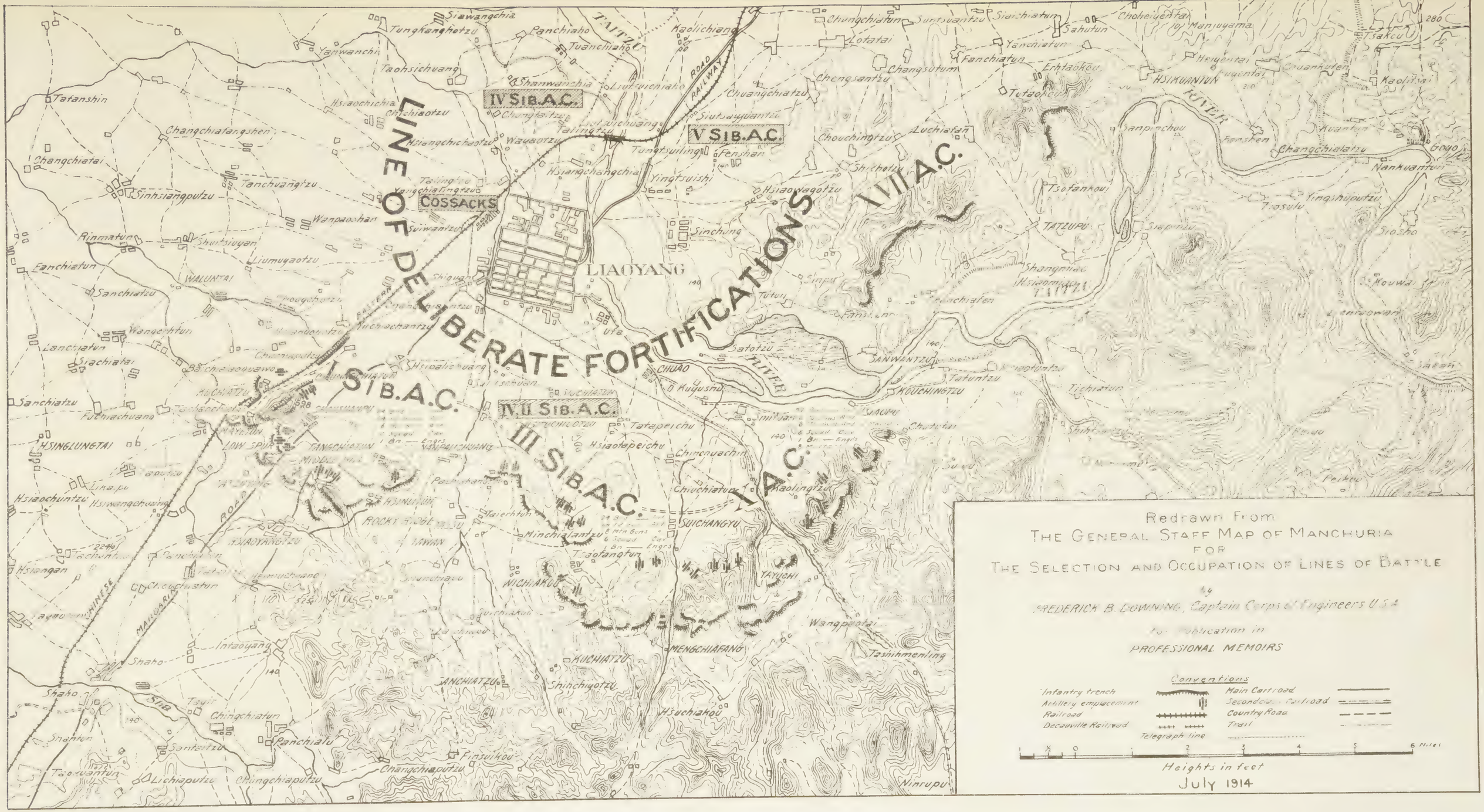
a. Lieutenant General Zarubaev, Fourth Siberian Corps, Second Siberian Reserve Division, First Siberian Artillery Brigade, Third Siberian Reserve Division, Second Werchneudinski Cossacks, Fourth East Siberian Sapper Battalion (28 battalions, 32 guns, 6 sotnias, 1 sapper battalion), will take position at the villages of Hsituchiaotzu and Siuchiatun.*

b. Lieutenant General Zasulich, Second Siberian Corps, Fifth East Siberian Rifles Division, Fifth East Siberian Rifles Artillery Brigade, Second Brigade Seventy-first Infantry Division (Two hundred and eighty-third and Two hundred and eighty-fourth Regiments), Seventh and Eighth Batteries Twenty-eighth Artillery Brigade, Seventh Siberian Cossacks (2 sotnias), Third East Siberian Sapper Battalion (20 battalions, 48 guns, 8 machine guns, 2 sotnias, 1 sapper battalion), will also take position at the villages of Hsituchiaotzu and Siuchiatun.

c. Lieutenant General Dembovski, Fifth Siberian Corps, Fifty-fourth Infantry Division, Twenty-sixth Artillery Brigade, Seventy-first Infantry Division, Twenty-eighth Artillery Brigade, Fifth

*This is quoted from the translation given in U. S. Reports of Military Observers. But the German reports state instead: the IV corps "will be stationed behind the northern wall of the town."


Fanchiafen
Tatuntzu
Su-yu


LINE OF DELIBERATE FORTIFICATIONS
LIAOYANG
I SIB.A.C.
II SIB.A.C.
III SIB.A.C.
IV SIB.A.C.
V SIB.A.C.
I A.C.
IV A.C.
VII A.C.
COSSACKS
TAITZU
RAILWAY
RIVER
Redrawn From
THE GENERAL STAFF MAP OF MANCHURIA
FOR
THE SELECTION AND OCCUPATION OF LINES OF BATTLE
by
FREDERICK B. DOWNING, Captain Corps of Engineers U.S.A.
for publication in
PROFESSIONAL MEMOIRS
Conventions
Infantry trench
Artillery emplacement
Railroad
Decauville Railroad
Telegraph line
Main Cartroad
Secondary Cartroad
Country Road
Trail
Heights in feet
July 1914

East Siberian Sapper Battalion (24 battalions, 80 guns, 1 sapper battalion), will post a detachment north of the village of Chuao, the rest of the corps will take post on the right bank of the Taitzu.

d. Major General Samsonov, Ural Cossack Brigade, Siberian Cossack Division (Grekow), Second Brigade Orenburg Cossack Division, First Argunski, Third Trans-Baikal Cossack Horse Artillery Battery, Tenth Horse Artillery Division (Eleventh and Twentieth Batteries) (48 sotnias, 18 guns), will take post east of the village of Yangchialingtzu in rear of the lines of forts.

III. PROTECTION OF THE FLANKS.

I confide the protection of the left flank to General of Cavalry Baron Bilderling.

The right flank to—

a. Major General Mishchenko, Trans-Baikal Cossack Brigade (11 sotnias, 6 guns). He will stand ready at the village of Wuluntai to protect the right flank of the army westward from the railroad to the Taitzu River on the line Hsinglungtai-Dawan and maintain communication with the detachment at Hsiaopeiho.

b. Major General Grekov, Orenburg Cossack Division, First Orenburg Cossack Brigade (12 sotnias). He will cover the right flank of the army on the right bank of the Taitzu.

IV. The artillery of position will be posted as directed by the inspector general of artillery.

V. The line of forts west of the railroad as far as the right bank of the Taitzu River and Fort No. 8 will be occupied by the Irkutsk Regiment under the direction of Major General Maslov, commanding the fortified rayon of Liaoyang.

VI. The chief dressing station will be at the Liaoyang railway station.

VII. The flying artillery park brigades will be echeloned in each corps by the corps commander conformably to the special instructions governing them.

VIII. The trains will be parked along the roads assigned to the corps. Their protection will be regulated by corps commanders.

IX. Reports will first be sent to headquarters at Liaoyang which will be connected with the stations of corps commanders by telegraph and telephone. Corps commanders will cause this connection to be made. Each army corps will furnish two orderly officers to the chief of staff of the army.

X. Corps commanders will designate the roads by which the troops are to march into their positions. These roads will be improved, provided with signposts, and made known to the troops.

XI. Lieutenant General Sakharov and General of Cavalry Baron Bilderling will act as my deputies.

XII. The positions will be occupied under the direction of the corps commanders.

It should be noted in brief that the above order gives the following:

1. The general situation and the purpose of the army commander.
2. The sub-divisions of the battle front into limited sectors, and the assignment to each sector of a commander and troops.
3. The assignment of troops to, and the location of, the general reserve.
4. The assignment of troops for the protection of the flanks.
5. The disposition of the artillery.
6. The location of the principal dressing station, instructions as to parks and trains, the commanding general's station, lines of information, etc.
7. It charges the corps commanders with the responsibility for the occupation of their sectors.

This order assigned General Stackelberg to the command of the Shoushanpu position. In the retreat from Anshantien to Liao Yang, Stackelberg visited Shoushanpu Hill, prior to his occupation of the position and studied it several days.

He, in turn, subdivided his front of battle into sectors, to each of which he assigned particular commands. "Nine battalions held the line from Kuchiatzu to the Hsiaoyangtzu-Shoushanpu road, under General Gerngross; six battalions extended the line from that road to the Hsiaoyangtzu-Hsinlitun road, under General Kondratovich; the Twenty-third Regiment was on the height of Fangchiatun; nine battalions were held as reserve under General Zikov at the village of Shoushanpu; eight squadrons or sotnias of cavalry were at Nanpalichuang under Colonel Gurko." The artillery was posted as shown on the accompanying map. Each officer assigned to the command of a certain subdivision of the line was responsible for its occupation and defense.

The following, taken from the British Officers Reports of the Russo-Japanese War, is a good description of General Stackelberg's sector of the front of battle of Kuropatkin's army at Liao Yang. The front of the rest of the army, though not here described, may be studied from the map.

The Shou-shan-pu position consists of three hills, joined together by cols, which are at no great elevation above the surrounding plain. The northwestern hill, called Shou-shan-pu, is a rocky eminence rising to a height of 693 feet, with steep sides verging in places on the precipitous. Its base forms an irregular triangle,

the longest side of which with buttress-like ends is hugged by the railway, which here and for some miles north and south is carried on an embankment 10 to 15 feet above the plain. From the summit of the hill, on which stand the still solid remains of an ancient beacon, an extensive view of the Liao plain is obtained. On every side, to the west and north, as far as the eye can see, and in other directions up to the foot of the mountains, is a huge expanse of kaoliang (tall grain) or millet, growing to a height of at least 10 feet, while here and there are villages of varying size hidden in clumps of trees and rising above the surface of the level ground like islands in the sea. Next to the hill of Shou-shan-pu is Middle Hill, which lies immediately east of the main road to Liao-yang, and is the central feature of the position. Its sides, though steep, are less abrupt than those of its neighbors, and consequently lend themselves better to defense by rifle fire; moreover, it stands somewhat in advance of the general line, and so flanks Rocky Ridge, the next hill on the east. Between Shou-shan-pu and Middle Hill, thrust out from the latter, is a flat-topped underfeature—Low Spur—which not only commands the approaches by the main road, but serves to flank the front of the hill from which it springs as well as that of Shou-shan-pu. On the neck between Middle Hill and Rocky Ridge, and lying somewhat in the background, is a small knoll, well hidden from the plain in front and overlooking the stem of the deep and Y-shaped ravine before it. The last hill of the series is Rocky Ridge, with long and narrow top, stony, steep, and in parts thickly covered with low bushes and scrub. Bulging out on its southern side is a knoll which flanks both the front of the parent hill and that of Middle Hill. This knoll in turn is flanked from the ridge behind as well as from the sides of Middle Hill.

Although the range runs on beyond the road to Hsin-li-tun, that portion was not included in the Russian position, which from Rocky Ridge, bends northward along the crescent of hills overlooking the last-named village. These hills effectually guarded it on the east, while the railway embankment did similar service on the west. The whole position is in shape a rough horse-shoe, of which Middle Hill is the center of the curve, and the railway and hills above Hsin-li-tun are the sides. Nearly at right angles to it and facing the eastern extremity of Rocky Ridge are two narrow valleys whose full significance will appear as the engagement is described. Apart from artificial aids, the ground which the Russian right held was as strong as could well be imagined, though, like all positions, it was not perfect.

The strength of the left was somewhat detracted from by the lack of depth of Rocky Ridge, while the bastion-like positions of Low Spur and Middle Hill made the works upon them liable to enfilade and even to reverse fire from long-range guns posted beyond the Russian right, north of the railway line. As regards the latter contingency, the sodden nature of the ground at the time of

the battle would have made the movement of heavy ordnance, except in trucks upon the railway, extremely difficult, so that little danger was to be expected from that direction.

On the whole, the Shou-shan-pu position was admirably placed to serve the Russian purpose of safeguarding Liao-yang from the south and west. It covered that city and the bridges over the Tai-tzu Ho at suitable distances, and its great natural strength did not demand an exceedingly large garrison. It commanded the approaches, and the field of fire along the whole front was extensive, the tall millet which would have obstructed the view from the lowest trenches being levelled or broken down to the front for about one thousand yards. The flanks were secure and rested on the inner line of works near the city; excellent cover for reserves existed behind the hills, and the cols between them so hid the Russian artillery, which made use of indirect fire, that neither by day nor by night could the flash of a gun be seen. Communications were all that could be desired; the ground on the right was favorable for counter-attack, while the line of retreat led by many roads across a plain. Even water, often a difficulty where large forces hold small spaces, was easily procured from springs and wells close by. Lastly, from Shou-shan-pu the position of nearly every Japanese gun, though hidden in the kaoliang, could be noted, and where this was not possible, could be observed by means of a balloon.

From the above description it will perhaps be apparent that the Shou-shan-pu position was not deficient in natural defensive power, and in order to enhance that power the Russian engineers turned to account nearly every device which forms part of fortification in the field. Here, in strict contrast to the battlefields of Nan Shan and Te-li-ssu, were no conspicuous gun epaulements nor trenches visible to the naked eye for miles; yet by their appearance the works must have been constructed many weeks, probably months, before the attack.

The general character of the works at Shou-shan-pu and the adjacent hills may best be described as semi-permanent. They were prepared for infantry to hold one or two lines of trenches, both placed well below the crest-line, with their parapets sod-covered or weed-grown, which merged gently into the slopes in front. On each hill, from one or other extremity, and sometimes from both ends, of these lines, deep and well defiladed approaches led to the splinter-proofs behind, or to the ground immune from gun or rifle fire.

From west to east the defenses were as follows:

Beyond the railway a strong line of shelter trenches protected the embankment from sudden attack at the point where it curves, while another line connected the village of Ku-chia-tzu with the permanent way. The existence of this embankment was fortunate for the defense, for being raised above the plain it afforded a good field of fire and protection against artillery, while on the west of it

was a ditch filled with rain water, whence the earth required for its construction had been taken. Four hundred yards southwest of the point where the railway bends, and flanking it, a Russian cottage with outhouses was prepared for all-round defense with strongly-made parapets backed by sleepers. Round that part of Ma-yeh-tun village which is nearest to the railway was a blunted lunette with thick parapets and a 10-foot V-shaped ditch. This work was incomplete, there being no ditch on its western face and no obstacles in front of it. The walls of the rest of the village had been prepared for hasty defense, and to the east of it ran a trench which partly flanked the works on Low Spur. Below the crest of Shou-shan-pu were cunningly made breastworks built of rock and sand-bags, and on its flank a trench from which fire could be delivered to front or rear. Communication with this hill was not forgotten, for, from near the summit, a double flight of easy steps was cut, which led to the plain in rear through a shady grove, on the verge of which stood a Chinese temple. Throughout the battle the deep-tongued bell of this edifice tolled the hours, regardless of din of gun and rifle. On the col south of the hill was a completed work of nature similar to that before Ma-yeh-tun. Low Spur was held by an inconspicuous trench from which three covered ways conducted to a roomy splinter-proof, behind which lay a deep and narrow ravine.

The defenses on Middle Hill were much like those on Low Spur. Just below the summit was a circular trench, and lower down the hill side a long line of trenches stretching across the face from west to east. Behind the southern top were splinter-proofs connected with the trenches, and further cover was available in the deep and broad ravine at right-angles to the main road. On the neck between the two peaks of the hill were gun pits for four guns, which flanked the front of the whole position to the east. The rough crest of Rocky Ridge provided natural cover for infantry firing towards the south, but here and there, more especially towards the eastern extremity of the hill, were trenches, some of which were thrown back to meet a possible attack on the flank. Well below the crest, and running partly round the knoll before it, was another trench, whose fire swept the ravine in front. Approaches joined this trench to splinter-proofs behind the ridge, but neither these nor some behind the knoll were in a completed state.

The greater part of the front of the position was closed by a broad line of high barbed wire, or rows of military pits, or both in combination, but wide gaps were left in front of Middle Hill, and where the roads penetrated it there were no obstructions. The obstacles, which were completely under view from the trenches, were situated from three to six hundred yards in front of them. On the Russian left the entanglement was carried to the eastern side of the road to Hsin-li-tun, whence turning northwards it ran through the neck parallel to it for a short distance. On this flank,

too, west of the road and on the lower slopes of Rocky Ridge, was a row of stone fougasses which could be fired by means of an electric wire from the position.

Sites suitable for hiding the Russian guns were found behind the cols, where in several places alternate lines of epaulements were made. The locality of two of these artillery positions deserves remark, for to them was greatly due the extreme difficulty which the attackers found in gaining a lodgment on Rocky Ridge. Of these, one lies behind the rearmost peak of Middle Hill, from which point the fire of its guns played upon the exits of the narrow valleys leading north from Yen-tao-yuan. The other was on the slope of the hills near Fang-chia-tun, whence the guns fired directly up these valleys, their projectiles crossing those from Middle Hill.

To complete these elaborate arrangements, a light railway, starting from near the railway station at Liao-yang, was laid through the fields to the gun position at Fang-chia-tun, whither it carried ammunition and supplies.

Such was the obstacle which faced the Second Japanese Army—one which held it at bay for forty-eight hours of almost ceaseless battle.

The Second Japanese Army attacked Stackelberg's position on the morning of the 30th of August, and continued the attack all day. On this day two unsuccessful assaults were made on the trenches near the railroad. At dawn on the 31st the Japanese made another assault on the position between Low Spur and Rocky Ridge hills. Again the assailants were driven back. About noon, a fourth assault was made. Some of the advance trenches were captured and held. About dark on this, the second day, the 31st of August, 220 guns were brought into action against the Shou-shan-pu position in preparation for another assault. But shortly afterwards, Kuropatkin, having been informed that his flank had been turned, decided to retire. Stackelberg was then ordered to fall back to the inner defenses of Liao Yang. Splendid courage was shown by the Japanese in their assaults of Shou-shan-pu. The Second Japanese Army lost in killed and wounded more than seven thousand men, but Stackelberg held his position as long as his commander desired.

The enormous length of modern battle lines and the limited time for their preparation for defense, precludes the possibility of the corps commanders fixing the lines of defense, except in a very general way. The front of the position at Liao Yang was 20 miles long; that at Mukden 50 miles long.

To the brigade and regiment commanders must be left the disposition of their troops and the approximate selection of the line

of battle. The final siting of the defenses will be delegated, usually, to the officers of the line serving with troops, to the majors of battalions and the captains of companies. The duty thus imposed exercises the skill and training of the officer to a high degree. Probably no more difficult duty nor one upon whose proper performance so much depends, falls to subordinate officers.

The higher commanders in determining the approximate locations of the line, base their selection upon the study of maps. Their selections, however, can be only approximate. The positions finally occupied usually depart from the apparent possibilities of the map selections, no matter how large the map scale. Too much emphasis can hardly be given to the fact that the best way to fix the location of the trenches is actually to outline them on the ground, marking the tentative crests with stakes or men, and moving these to the front or rear until the line is finally established. Practice in this work is invaluable. At 203-Meter Hill, in the defenses of Port Arthur, a trench that had been laid out by an engineer officer and constructed in solid rock at the cost of great labor was upon its occupation found to be poorly placed. It was abandoned and another trench a few yards distant was constructed by the troops.

The troops to be assigned to the position govern the selection of the line. Those to be considered first are infantry and artillery. The other troops may be considered as auxiliaries of secondary importance without effect upon the selection of the position.

Machine guns are employed in the infantry trenches. They are especially desirable in the salients and on the flanks, and where for any reason it is desirable to develop a great volume of fire. They do not affect the selection of the line of defense.

The cavalry performs its duties in the service of security and information. It harrasses the enemy in flank and in rear, or it is employed on the flanks as a mobile guard. It entrenches as does the infantry.*

Formerly the two arms, artillery and infantry, were placed in juxtaposition in the defenses, but experience and improvement in the use of artillery has shown that the best results are obtained when the infantry sites and the artillery positions are separated, but so related that the cooperation of these arms is still maintained.

The fighting range of heavy artillery extends to 6,000 yards;

*The Russian cossacks at Liao Yang were not well placed.

whereas, infantry fire can not usually be effectively employed beyond 1,000 yards. Artillery uses indirect fire most advantageously—infantry can not employ it. The best target for artillery is material, or troops in close formation; the infantry target is usually a single skirmisher, with probably only his head in sight. Good artillery sites are not usually good infantry sites, and vice versa. The above and similar considerations have conduced to the use of separate sites for the two principal arms.

The artillery consists of field and siege pieces. It should be placed behind the infantry firing line a distance between 200 and 1,000 yards. A shorter distance would subject the infantry line of the defense to the hostile fire of the enemy's artillery when directed against that of the defenders; a longer one would too greatly diminish the distance in front of the position covered by the defender's artillery. It will usually be possible to utilize the natural features of the terrain to great advantage in securing concealed and protected positions for the artillery sites, and thus decrease the necessity for the construction of artificial positions. In the Russo-Japanese War, however, positions were almost invariably prepared before the guns were brought into action. Simple types were used, elaborate care being given to concealment. When time permits, the natural positions should be improved by the construction of gun-banks, pits, emplacements, bomb-proofs, magazines, communications and observation stations. But a part of the artillery may be held in readiness to take up its firing position when the enemy develops his attack. The artillery is used against the enemy's artillery; against his positions in mass and against his infantry advancing in dense formation over the routes of approach; against his troops in dense attack; and against his captured positions. It should cover all of the approaches to the position, both to the front and to the flank. It will be usually arranged in groups, by batteries. For protection, these groups may best be separated; this is also true of the several guns of the batteries.

Skill, experience, thoroughness and persistence are essential qualifications of the officer responsible for the location of an infantry line of defense. These qualifications should be trained. The art of fortification is based upon simple principles that can be mastered by the average intellect. But this knowledge if unused is lost. Long practice produces skill. Let it be remembered that in their war with England, the Boers, unlearned in the text-

books of fortifications, practiced those very principles which the books often obscure with a mass of detail. And, let it be remembered also, that in peace not in war should the duties of war be learned.

Experience shows that the defense of the infantry position is best made by concentrating the defenders along a single line of defense. This main line must be so strongly fortified and defended that an inner or reserve line would serve no useful purpose. The logic of this appears upon consideration of the effectiveness of modern weapons and the purpose of fortifications. For, should the outer line be carried, the enemy can then place his big field-pieces so that his bombardment of the fortified center will easily reach the vulnerable points thereof and shortly compel its surrender.

Moreover, it appears that if the defenders, in the stubborn defense of a well-selected, well-fortified main line, can not compel the enemy to abandon his attack or can not thus gain sufficient time to obtain the necessary reinforcements with which to take the offensive or to further resist the attack, they must exhaust themselves and render hopeless the resistance of an inner line. Furthermore, with successive fortified lines in rear, the troops will be prone to yield their positions, lose their offensive spirit, and fall back before the most desperate assaults of the enemy. They will fight more bravely when they are emboldened by necessity and brought to bay.

The complete reliance of the defense must generally be placed in a single line. But in the land defense of seacoast fortifications an inner line is necessary. This inner line, however, is by no means a second position for the defeated troops of the main line of defense. It is a line of infantry fortifications that is built around the rear and flanks of the seacoast batteries at convenient distances, say 100 to 1,000 yards therefrom. Its purpose is the defense of the emplacements from the sudden attacks of raids successful in landing within the flanks of the main line, or against small bodies of the enemy who, by stealth or through local temporary success in carrying or evading the main line, assault the coast forts from the rear. This line of immediate defense, as it may be called, is prepared for defense by machine guns and small field guns. It consists of infantry trenches or ordinary stockades and obstacles entirely closing the rear of the fort. It may properly be considered a part of the seacoast fort. Whenever practi-

cable without unduly lengthening the inner line, it should include such accessories as search-lights, range towers, power-plants, etc. Its location is best determined when the batteries and their accessories are planned.

The inner line of the Russians at Liao Yang did not constitute a second line of defense. It was primarily a bridge-head, which protected the Russian retreat across the Taitzu River when they abandoned their defense of the position in front of Liao Yang.

The complete reliance of the defense must be placd in a single line. This line, the main line of defense, constitutes the fighting line. It is manned by infantry or troops employed as infantry and by machine guns. Usually the battalion, under its major, will be assigned a definite sector in the line of the regiment. The major fights his battalion. The colonel fights his regiment. Each organization provides its own reserve, that of the battalion is usually placed in the cover trenches immediately in rear of its fire trenches; that of the regiment, in cover trenches farther to the rear, located to reinforce any part of the regimental line; that of the brigade where its commander decides, and so on up the scale. For tactical and economical reasons, the line of defense connecting the battalion positions is not unbroken. Nor are the firing trenches of the battalion placed in an unbroken line. The fighting line therefore consists of a series of localities organized and prepared for defense, connected at intervals by a broken line of trenches. (See map of Liao Yang position.) The battalion defends its special part of this line. When the position is prominently defined by the topography, its fortification is variously called a redoubt, a supporting point or a ring trench. But the battalion position forms an integral part of the fighting line. The major is responsible for its defense. It is divided into the firing line, the supports and the local or regimental reserves. The firing line occupies the fire trenches; the supports the cover trenches; and the local or regimental reserves, sheltered positions in rear.

The firing trenches are built in sections appropriate to the size of the company or platoon or squad that is to occupy them. The subdivision of the line into squad lengths is convenient. But usually the trench lengths will be occupied by a platoon or a company, or will be of a corresponding length. Long trenches should be avoided. Ample provision for sheltered travel between the parts of the fighting line should be provided.

The captain is responsible for the defense of the front and the

flanks of his company. He mans his fire trenches as occasions require. His guard remains in them, and except in emergencies is reinforced from his cover trenches and by his own supports only. The major however conducts the defense of the battalion position.

The term field fortification is applied to works thrown up by an army in the field to increase its chances of success in impending operations. They are usually the result of immediate necessity, which might or might not have been foreseen and provided against by permanent defenses. They are not so deliberately planned or executed as the works they substitute, but they serve the same end and are built in accordance with the same principles.

Much has been written of the use of field fortifications in the Russo-Japanese War, of their remarkable development in the siege of Port Arthur, and of their use by both the Russians and the Japanese in the field. The increased importance of protection against the fire of modern weapons was emphasized by the war; but in obtaining such protection, no new principles were developed. However, the relative weights of the considerations affecting the construction of defenses was modified. Concealment became of prime importance. Among other modifications to be noted are: the necessary provision for desperate fighting at point-blank ranges and for the repulse of assaults by night; the necessity for a practically continuous and impregnable line of defense with no undefended intervals, and the conflicting consideration of holding a line, points of which an enemy has captured. The use of machine guns, lines of communication and information (railroads, telegraph, telephone, roads, trenches, etc.), and of obstacles, increased in importance. The great value of machine guns was appreciated. Ample provisions were made for their employment.

The broad principles of fortification do not change, but the continuous development of the art is in rapid progress, keeping step with the improvements in ordnance and the conduct of battle. Recently, in Professional Memoirs, arguments were advanced for certain forms of redoubts, thus bringing into debate a type of redoubt, treating it as an independent work, more or less isolated. There need be no discussion of such stereotyped forms for use in the main line of infantry trenches. The redoubt, so-called, should be considered merely a constituent part of the line of trenches. There is a type trench, but there is no type redoubt.

It is the intention here, not to deprecate the use of supporting points, but to emphasize the error of a tendency to search for a

type redoubt and to treat it as an independent work. Type redoubts are impracticable—better completely abandon the idea.

In support of these statements it may be well to quote here some opinions that are based on observation of recent wars and the study of the use of intrenchments. They fairly state the truth.

Colonel Stuart—

It is thought that a correct conception of the function of a redoubt in a defensive line is quite as important as the form of the redoubt itself. In my opinion no redoubt should ever be built unless it figures as a part of some defensive line and can be seen in its proper relation to the other elements of such line.

Major Harts—

Supporting points of the present day may be said to vary from the few inconspicuous groups of trenches of the German Field Regulations on the one hand, to the elaborate Russian redoubts on the other, where little concealment is afforded, but instead, a prominent target congested with troops.

And further,

On varied ground of considerable relief only the general principles can be fixed, as the details must conform to the site and it is only on comparatively level ground that our field work will ever approach a type design.

Major Flagler—

For supporting points, fortified pivots or whatever term is applied, in a defensive position taken by a field army and strengthened, it is preferable to fortify at intervals the strongest points afforded by the terrain with a carefully devised system of trenches, obstacles, bomb-proofs, etc., adapted to each individual site. Such fortified points will almost never present a symmetrical work.

In the same way might be multiplied the opinions of military engineers that the idea of completely closing the trench of the supporting point is not often the best way to fortify the position. But let this opinion suffice.

Colonel Kuhn—

In the rare cases that a closed work may be used, such as a support for a wing or behind (as at Liao Yang), or in an especially weak or dangerous front, it will ordinarily take the form of a simple ring trench inclosed by a continuous line of obstacles and will have a fixed garrison assigned to it.

Frequently in the interest of concealment, and especially on hillside positions or on exposed sites, it is better to use tiers of trenches

which are successively retired, as it were in echelon, than to attempt to close the work by abruptly leading the flanks up the slope of a steep hill, sacrificing concealment without securing a fire to the flanks. It is very true that a general type of trench is desirable and attainable, but such is not true of a general type of redoubt. The question as to the location of the trenches and the trace of the work is in every case to be determined as a special problem.

Advanced posts in front of the main line of defense sometimes serve to delay reconnoissance by the enemy, and to cause him to expend time and men in their capture, and thus early to disclose his positions. But their use in general is to be shunned. They ultimately give a lodgment to the enemy.

If in selecting a line it appears that advanced posts are advisable, a good test to apply is this: imagine the main line of defense pulled forward to pass through these positions. If this location of the line is as good as the one in rear, place it here finally; otherwise give up the advanced position. In laying out a line the tendency is usually to go forward. Every irregular hill and slope tempts the defenders forward, only to find that still farther to the front are tempting sites for defense. Let it be realized that in the line to be occupied there are certain well-defined points and that upon these with connecting trenches must be hung the chain of defenses. Examine all promising positions in front but do not lose sight of the controlling features. And if advanced posts are used, the trenches on them when captured by the enemy should not assist him in his attack on the main line.

The positions finally selected for the defense should utilize to the best advantage the existing features of the locality. To this end the ground should be closely studied. Account should be taken of all natural and artificial objects and conditions: among these are hills and valleys; ridges and plains; lakes and rivers; streams and ponds; ditches and hedges; woods and villages; houses and fields; walls and inclosures; roads and railroads; bridges and dams; cuts and embankments; swamps and marshes; rock and earth; vegetation and drainage. These are to be economically utilized to secure:

1. An unobstructed field of fire in front of the line of defense.

2. A shelter which will protect the defender from the assailant's fire, and a mask which will conceal him from the assailant's view.

3. An obstacle by which the advance of the assailant will be retarded or directed.

4. Easy communications for the tactical movements of the defender.

5. Obstructed communications for the movements of the assailant.

These requirements remain unchanged by discussion. The first of them, the unobstructed field of fire, is obtained primarily by the proper location of the firing crest of the parapets or trenches and by clearing the foreground within the limits of the field of fire, as far as practicable, of all woods, fences, buildings, etc., that may afford shelter or concealment to the enemy. Such clearings necessitate a great amount of labor and consume a large part of the time available for the preparation of the defenses. Where the débris therefrom can be used in the construction of other parts of the defenses, such as screens, bomb-proofs, splinter-proofs, shelters, and obstacles, such as wire entanglements, abatis, slashings—the clearing is doubly useful. Where no such use can be made of the débris, that not necessary for fires, cooking, etc., may be burned in place. The width of the clearing should be at least 800 yards if time permits.

No amount of clearing, however, will compensate for an improperly located trench. Usually no continuous single line of trenches can be found without certain dead spaces. It is essential that these dead spaces be provided for. This may be done by covering them with the fire from neighboring trenches located for this purpose, or from advanced or retired lines built with the same object; or, they may be obstructed with obstacles.

Positions near the foot of a slope, and at or below the military crest, are favorite ones for the trenches. High command is not essential, sky-lining is especially to be avoided. Grazing fire from rifles is the most effective; flat fields of attack the most deadly.

The notes on field fortification by the Army Field Engineer School are largely quoted immediately below. They have been somewhat modified by the writer.

If it were merely a question of obtaining a good distant view, the best location for fire trenches would unquestionably be on the military crest of heights with rather steep slopes, but such a location involves two disadvantages, first, the sacrificing of grazing fire, which is much more effective than plunging fire, and second, the exposure of the trenches to the enemy's view. If the slopes of the heights are too abrupt, there is further danger of dead spaces at the shorter ranges.

Fire trenches located upon prominent crests are on the whole disadvantageous, offering good targets to hostile artillery, which will thereby be able to support the advance of the hostile infantry with good effect. Such locations are inadvisable unless the trenches be so strongly constructed that they can withstand ordinary artillery fire, such as the Russian trenches on 203-Meter Hill. In locating a trench upon a crest, care must be taken that it be not outlined against the sky. To avoid this it will be necessary at times to push the trench some distance down the forward slope. If the forward slope is convex in profile or consists of a succession of crests, it will sometimes be necessary to advance the trenches well forward in order to secure the best field of fire.

Positions forward of the crest sacrifice command and limit distant view, but are less exposed to hostile fire. The chief disadvantage of a position forward of the crest is the difficulty of connecting with the fire trenches either for reinforcing the same or for withdrawing. Such being the conflicting considerations, it is manifest that the location of rifle trenches with respect to the foreground must be a matter of compromise to be determined by sound judgment on the part of officers responsible for their location, usually officers of infantry.

To secure the advantages of a good view and at the same time to minimize the exposure of the trenches, a location on gently rising ground having a concave slope is unquestionably the best. But the terrain must be taken as we find it. Ideal positions are seldom found.

Often, in close country, it will be impossible to obtain a fire-swept zone in front of the trenches wider than 200 or 300 yards. But this zone can not be crossed if the defenders' trenches are well sited. In such places several tiers of trenches may sometimes be useful in order to increase the volume of fire, but there should be no idea of using these trenches as successive lines of defense. Here, also, machine guns are especially useful in increasing the volume of fire. Obstacles must be judiciously used, to force the enemy into those approaches that are best defended by fire and to obstruct the others.

The defense of the position is finally accomplished by the fire of the defenders. They can not be driven from their trenches by artillery. The enemy resorts to infantry assaults. Then concealment is no longer necessary. In the prolonged struggle for the

field work on 203-Meter Hill the hand grenade, the bayonet and the butts of rifles even, were used.

The fire trench that will usually be constructed at the first occupation of the position is that shown in Fig. 1.

Fig. 1 is copied from the Field Service Regulations. It is also shown in the Infantry Drill Regulations. In average soil it can be constructed with portable intrenching tools in one hour and a half. "Slight modifications may at times be desirable or necessary; to secure a somewhat higher parapet to better enable the foreground to be overlooked; to suppress the parapet for better concealment; or to provide for the contingency of difficult soils preventing a depth of trench as great as 3 feet 6 inches." These modifications result from raising or lowering all of the parts of the type trench an equal distance above or below the surface of the ground—that is, in the figure the plane of site may be considered to be lowered or lifted. The dimensions given above are not to be accurately adhered to. They are approximate. The soldier varies them to suit his stature.

If the position is held for some time, the simple trenches will be improved, widened and strengthened and provided with sheltered trenches connecting the supports in rear. The simple trench may be thus developed into the complete trench shown also in Fig. 1, by wasting the earth indicated by cross-hatching. This work may be accomplished with the intrenching tools in about two hours.

The Russian trench used in front of Liao Yang is shown in Fig. 2. To complete this trench in average soils with full-size tools would require about ten hours.

Concealment of the position is desirable in the early stages of the battle. This is accomplished by assimilating the trenches to their surroundings so that the trenches blend with the site. Natural masks are best, and these must be utilized or they must be imitated by the defender with trees, bushes, boughs, etc. As a rule, the lower the parapet crest, the better the concealment. The present tendency is, therefore, to low parapets with flat superior slopes. Sharp angular changes of direction accentuate the shade and shadow effect of light, hence, angles are replaced by gentle curves. Fresh earth is easily visible at long distances. It is hidden by sod, branches, bushes or trees, when the immediate vicinity is naturally so covered—that is, the parapet should always be made to appear like the natural ground. Fortifications are least visible when their traces follow the natural features of the terrain or conform to the contours of the hills.

In connection with concealment, the use of head cover improvised by the defender, after the completion of the trenches, is here noted. A shield of steel, sandbags or logs, or whatnot, unless carefully used, when placed on the parapet gives to it a serrated appearance, which may completely disclose the position. Head cover should be provided when the trenches are built, so that the crest presents an unbroken line. It may be noted here also, that the use of dummy trenches sometimes may deceive the enemy and for a while effectually mask the real defenses.

The use of obstacles has long been recognized as of prime importance. They may be used: to obstruct an avenue of approach; to hold the enemy in a deadly area or to close weakly defended

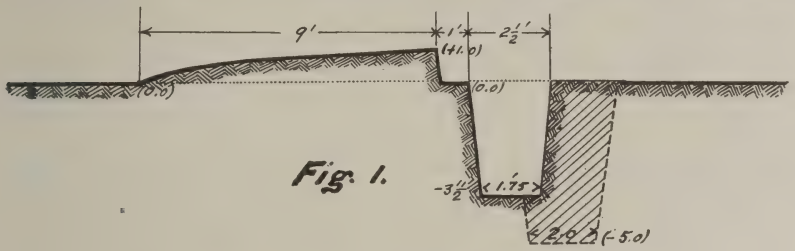


Fig. 1.

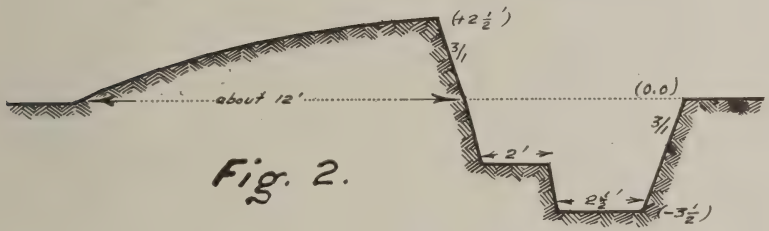


Fig. 2.

intervals. In employing them their purpose should not be lost sight of.

Natural obstacles, such as streams, rivers, gullies, swamps and marshes are the best. Sometimes such an one can be produced by floods caused by the construction or destruction of dams. That obstacle for whose construction the material at hand is best adapted, is the one that should be used. Among artificial obstacles the high wire entanglement is the most extensively used. It is very effective, and when placed in a depression, it has the advantage of invisibility. Slashings and abatis are readily constructed in wooded areas. In the Russo-Japanese War, extensive use was made of all the text-book obstacles—palisades, fraises, crow-feet, military pits, chevaux-de-frise, ditches, mines, fougasses, abatis, slashings, wire entanglements, etc.

Fiebeger expresses well the use of obstacles as follows:

Obstacles are employed in connection with fortifications to protect the works from surprise, to break up the assailant's formations, and to hold his troops for a time under the accurate fire of the defender. They should be concealed from the assailant, they should neither give him cover nor conceal his movements, and they should be difficult to destroy. Obstacles may be placed either in front of or along the line of defense. If in front of the line, they are most effective if they are under the close infantry fire of the trenches, under close observation at night, near enough to the line of defense to compel the assailant to cease his artillery fire before his infantry reaches them, and far enough from that line to save them from destruction during the artillery bombardment which precedes the infantry attack. Obstacles along the line are either in the ditches of the fortifications or in the intervals between the works of the intrenched line. For passive defense the obstacles in front of a defensive line should be continuous; for active defense they are employed only in the defense of salients or key points.

They will be placed depending on the topography, say, 25 to 100 yards in front of the line of trenches. The usual distance will average about 50 yards.

Protected communications and trenches connecting the fighting line and the reserves and the artillery positions are necessary to facilitate the rapid covered movement of troops and material in the tactical defense of the front. A well developed system of roads is desirable for the supply and movement of troops and as routes for the transportation of ammunition, engineer material, subsistence stores, etc. In rolling or hilly country, these routes may be concealed by properly locating them in the folds of the ground. In flat or open country resort may be had to the artificial concealment and cover furnished by especially constructed trenches.

The Russian position at Liao Yang "was remarkable for the deep covered ways, which, twisting and turning so as to avoid enfilade, connected the trenches with strong splinter-proofs cut into the reverse slopes of the position."

Covered communications—that is, trenches and parapets, form parts of the works themselves, they also connect the separate works with each other and with the fire trenches, bomb-proofs and reserves. The masking of these passage ways is also of great importance. However, their concealment is not so necessary as that of the works themselves. They are used throughout the attack, but are most used during its later stages when the attackers have

advanced in strength against sections of the line, whose positions will have been more or less disclosed by the defenders, and when the ability to rapidly reinforce the endangered section is most essential. It is a general rule that men will fire at those trenches that return their fire, consequently trenches of communication that return no hostile fire are comparatively safe from attack. Moreover, when it is impossible longer to conceal the trench positions, time and labor consumed in fruitless efforts at concealment are lost.

The obstruction of communications for the movements of the enemy was discussed under obstacles. Under this subject may now be mentioned the destruction of railroads, roads and bridges in the enemy's zone of operations, prior to and during the continuance of the attack. The covering of the enemy's routes of travel by the indirect fire of artillery is also an effective means of obstruction.

Aeroplanes and balloons may be used in securing information as to hostile movements, and possibly successfully in obstructing them by the explosion of bombs. Air craft will operate usually in conjunction with the artillery communicating by wireless telegraph the enemy's movements and acting as observers to correct the fire of the artillery.

In this connection it should be noted that over-head protection and concealment are necessary provisions against the possible use of aerial observation and attack by the enemy.

The necessity for telegraphic and telephonic lines of information with ramifications to every part of the defenses need not be dwelt upon. The increased importance of such lines of information is evident upon considering that the fighting front may easily be 50 miles long. Wireless telegraphy will be used. Schwartz says there should be two sets of communication practically independent, "the first set should connect the works with each other and with the commandant; the other set should be for the service of the artillery, its principal object being to enable the chief of artillery to conduct the fire of his batteries." The risk of the interruption of these communications should be avoided, as far as possible, by the provision of reserve material for field telephone lines, etc.

The signal troops establish the lines of information.

The engineers are employed in opening roads, clearing the field of fire, constructing barbed-wire entanglements, setting up and operating search-lights, and in otherwise strengthening the position, in

building approaches, executing demolitions, and in mapping the area concerned.

The front of a division may be 5 miles long. One battalion of engineers (three companies) is assigned to the division. This will give, ordinarily, one company of engineers to the brigade. The company may be sub-divided into platoons, and one platoon under an officer may be assigned each regiment in the firing line. Further sub-division is possible, but it would lead to confusion and loss of time, and render the control of the engineer troops hopeless. Thus it appears that the supervision by engineers of the siting and construction of the trenches will be nominal only. These duties must fall largely to the infantry.

The writer copies here extracts from a translation of a Russian article written before the Japanese War. It applies so well to our own army that it appears to be directed against it.

During maneuvers a false direction has been given to the activity of engineers, usually they have carried out work which, according to regulations, should have been done by the other arms, or, they have been used as infantry companies, or finally they have been forgotten and have taken part in the operations without being of the slighest use to themselves or to the rest of the troops. The general impression of the engineers, gathered from work at maneuvers, is that their presence forms a source of embarrassment for the general, particularly in the solution of the question what to do with them.

The work that has been briefly outlined herein, as incident to the preparation of a defensive position is enormous. The lion's share falls to the infantry.

Before it is undertaken, the limitation of time should be appreciated. Ordinarily, the troops, under the supervision of their officers, will construct their own defenses. They should be spared all unnecessary labor. An officer, with unpracticed eye or untrained judgment, will waste the time of his command in arriving at a decision. If his selection of position is bad he will risk its loss and the defeat of the army. The work on the fortifications will be done, usually, in the following order: constructing the simple trenches, clearing the foreground and building the obstacles, building the cover trenches and the communications, providing the overhead cover, the splinter-proofs, etc., and improving the defenses. But no definite general rules for the order of the work can be given. The judgment of officers must be relied upon.

If too much be undertaken, or too little, or the order be faulty, disaster may result.

The writer believes that the reader may lay this paper aside with the thought "that it fairly states accepted facts and old principles. There is nothing in it that challenges debate, nothing that compels attention, nothing that is remarkable, nothing that is new.

"Is there one that can not understand it? No. One that can not grasp the principles that govern the selection and occupation of lines of battle? No. All understand them readily. They are simple principles.

"What further then is necessary? Nothing. It is enough to understand these simple principles. Under the stress and excitement of imminent battle, an intelligent though untrained officer, a colonel, a major, a captain or a lieutenant, with poor maps, or with none, would select the best positions possible, and occupy and strengthen them to the best advantage."

Just so well would he who but understands the musical scales play a difficult composition on a cathedral organ.

Henry Washington Benham.

BY

Lieut. R. W. CRAWFORD,
Corps of Engineers.

Henry Washington Benham was born in Cheshire, Connecticut, in June, 1813. He entered the United States Military Academy July 1, 1833, and was graduated four years later at the head of his class, when he was commissioned Brevet Second Lieutenant in the Corps of Engineers. His first duties were those of an assistant on the Savannah River improvements. In 1838 he was commissioned First Lieutenant in the Corps of Engineers, and was placed in charge of constructing the sea wall at St. Augustine, Florida. Afterwards, and until the outbreak of the Mexican War, he was engaged in superintending the construction and repair of seacoast defenses along the Atlantic Coast, among them those of Forts McHenry, Washington and Madison, and of Annapolis Harbor, Maryland.

In 1847, the outbreak of the Mexican War called him into active service. At Buena Vista, Benham was severely wounded and was brevetted Captain for gallant and meritorious conduct.* After the war, he returned to his duties on the Atlantic Coast, taking charge of important defense work in New York Harbor and of the construction of a sea wall in Boston Harbor. While so engaged he received his commission as Captain. In 1852 and the early part of 1853, Benham was in charge first, of the building of the Buffalo lighthouse, and then, of the construction of the Washington Navy Yard.

In 1853 he was called from his coast defense work and was made assistant in charge of the United States Coast Survey. He re-

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*On the twenty-fourth anniversary of the battle of Buena Vista there was published anonymously in the magazine *Old and New* an article by General Benham, entitled "Recollections of Mexico and the Battle of Buena Vista, February 22 and 23, 1847."

mained on this work until 1856, spending several months in Europe, traveling and studying, in connection therewith.

In 1855 Benham was commissioned Major, Ninth Infantry, but declined. In 1857 he again returned to seacoast defense work, being in charge along the New England coast, principally at Boston Harbor, and around Newport, Rhode Island. From 1858 to 1860, he superintended the construction of the fort at Sandy Hook, New Jersey, and the building of part of the Potomac Aqueduct.

The following year saw Benham again in active service. He was first sent to the Department of the Ohio, as Chief Engineer on the staff of Brigadier General T. A. Morris, where he took charge of fortifying the important Union base, Cairo, Illinois. In July, 1861, Benham again distinguished himself in action. He was in command of the engaged froces which defeated Garnett at Carricks Ford, West Virginia, for which service he was brevetted Colonel. In August of the same year he was commissioned Major in the Corps of Engineers, and Brigadier General of Volunteers. He took part in the campaign, under General Rosecrans, up the Kanawha River, West Virginia, against the Confederates under General Robert E. Lee, and commanded the leading brigade at the battle of Carnifex Ferry, September 10, 1861.

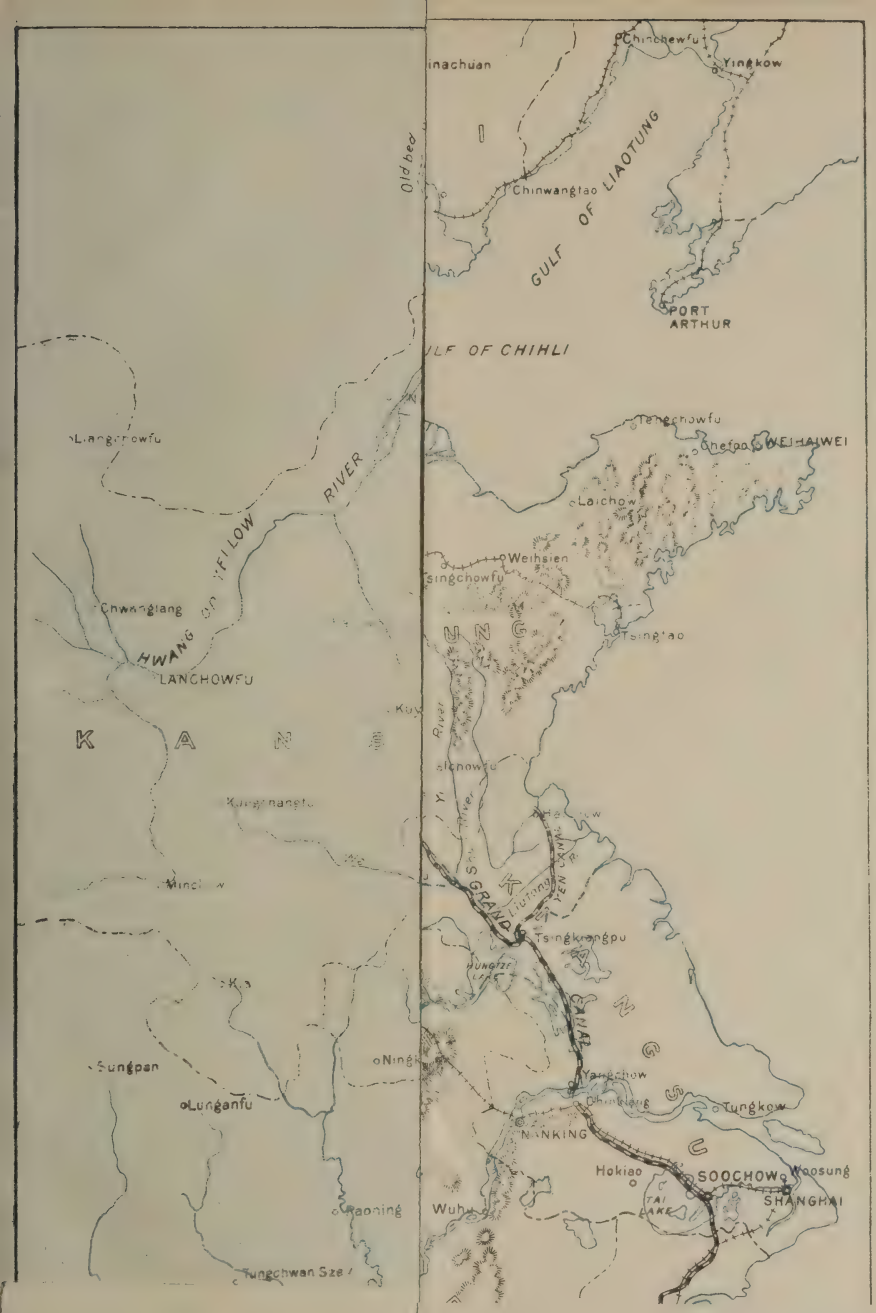
In March, 1862, Benham returned to the New England coast defense work around Boston Harbor but was soon transferred to take command of the Northern District, Department of the South. While in command here, he was present at Tybee Island, Georgia, during the bombardment and capture of Fort Pulaski, Georgia. The latter part of 1862, Benham returned to New England and superintended the construction of the fortifications of Portsmouth Harbor, New Hampshire.

From 1863 to 1865, Benham, then a Lieutenant Colonel, was in command of the Engineer Brigade, Army of the Potomac, except for a short period, when he was in command of the Ponton Depot at Washington, D. C. As commander of the Engineer Brigade, Benham superintended the construction of several famous ponton bridges, among which may be mentioned the bridge constructed, under fire, across the Rappahannock for the passage and retreat of Hooker's Army, from Chancellorsville, Virginia, and another, the most famous ponton bridge of the war, across the James River, at Fort Powhatan, Virginia. The latter bridge was 2,200 feet long—longer than any other ponton bridge ever constructed on

this continent. During this period, Benham also built and commanded the defenses of City Point, Virginia.

On March 13, 1865, Benham was brevetted Brigadier General, United States Army, for gallantry in the campaign leading to Lee's surrender. On the same date he was brevetted Major General, United States Army, and Major General, United States Volunteers, for gallant and faithful services during the Rebellion. He was mustered out of the volunteers in 1866, and the following year received his commission as Colonel in the Corps of Engineers. After he was mustered out of the volunteer force he returned to duty on the Atlantic Coast defenses, principally along the New England coast.

General Benham was retired from active service June 30, 1882, and died in New York City on June 1, 1884, at the age of 71 years.


Chinchewfu
Yingkow
Chinwangtao
GULF OF LIAOTUNG
PORT ARTHUR
Old bed
GULF OF CHIHLI
Tengchowfu
Chefoo
WEIHAIWEI
Laichow
Weihsien
Tsingchowfu
Tsingtao
SHANTUNG
Chwanglang
HWANG HO or YELLOW RIVER
LANCHOWFU
KANSU
Kuyuan
Kungchangfu
Mingchow
Kiaochow
Tsingkiangpu
GRAND CANAL
Yangchow
Chinkiang
Tungkow
NANKING
Hokiao
SOOCHOW
Woosung
SHANGHAI
TAI LAKE
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Sungpan
Lunganfu
Kaoning
Tungchwan Sze

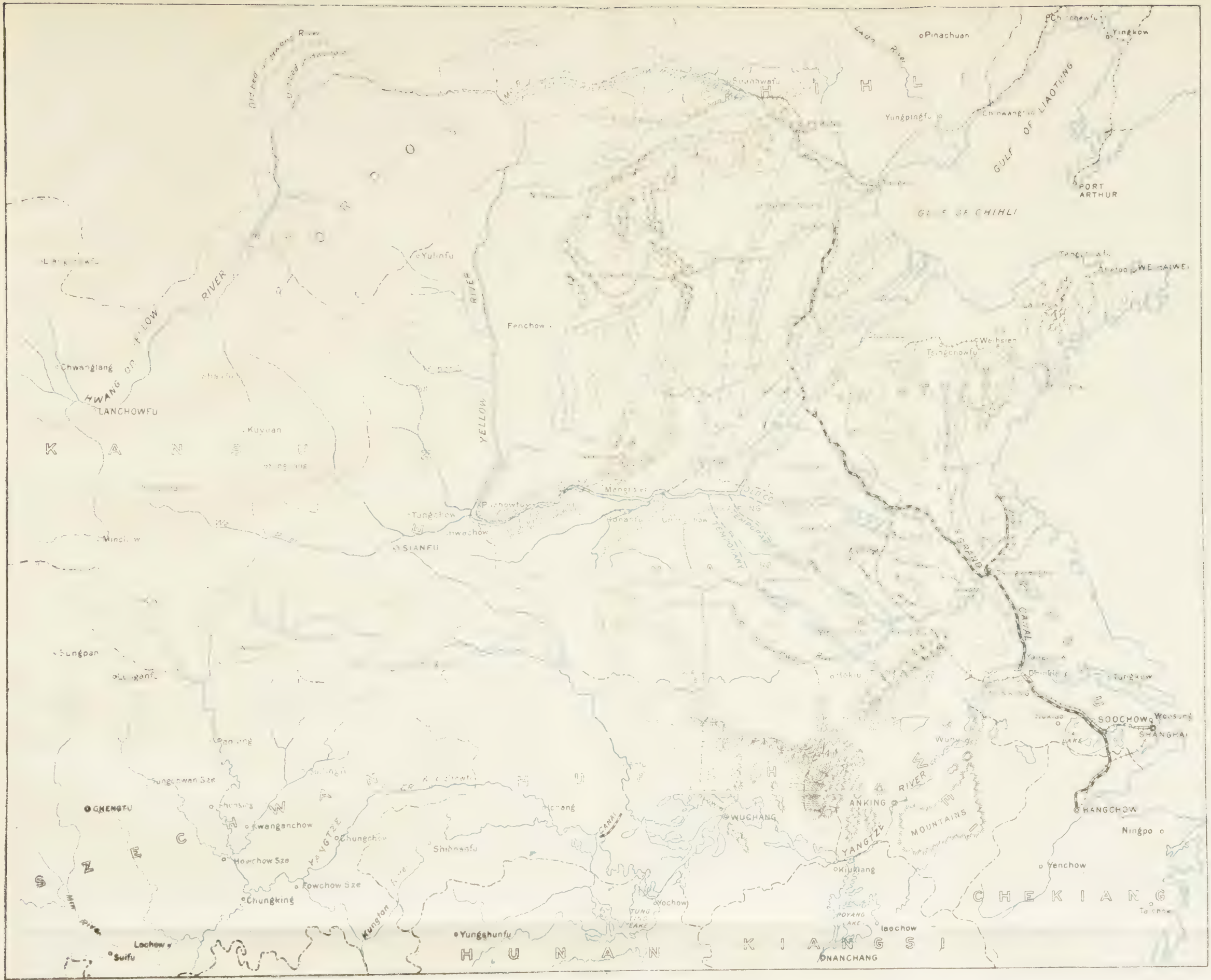


PLATE I

Map of Northeastern China.

The Huai River Conservancy Project.*

THE YELLOW RIVER AND THE HUAI DRAINAGE AREA.

With the exception of the extreme western tributaries, the Huai River drainage basin lies in the southern half of the great plain of eastern China. This plain occupies a roughly semi-circular area approximately 600 miles in radius around the eastern end of the mountainous Shantung Peninsula as a center. This great plain is of delta origin, developed in a great bay or partially enclosed sea, which in prehistoric times formed a western extension and union of the present Gulf of Chihli and the Yellow Sea. This basin was gradually filled by the deposits brought down from the surrounding mountains by the numerous streams draining them, including the Yangtse Kiang, the Huang Ho (or Yellow River), and the western tributaries of the Huai.

The Yellow River has been more largely instrumental in the formation of the Great Plain of China and in the creation of the abnormal conditions that obtain in the plain than any other factor. These conditions are due to its comparatively recent occupation of its present lower course, to the unusual physical character of its sediment, and to the conditions that surround its flow from the mountains to the sea.

From the point where the Yellow River is deflected south by a spur of the Yinshan to the point where it debouches on the great plains of China, the course of the river is comparatively modern. Of this part of the river Professor Bailey Willis (see Researches in China, vol. I, p. 234) says:

If we attempt to follow it from its delta on the great plain of eastern China, we have, indeed, a channel which marks a continuous river, but we have not a valley of erosion which we may attribute to the stream. As we saw it at the Tung-kuan, and as it is

*Extract from Report of Board of Engineers appointed by the American National Red Cross to make certain investigations. Received from Lieut. Col. Wm. L. Sibert, Corps of Engineers, U. S. Army, Member of the Board, and printed by permission of the Red Cross.

described in its course from that point eastward, it is a stream occupying a depression produced by normal faulting. It has taken possession of a channel, but has not made one. In its long course from north to south, above the great elbow between the provinces of Shan-si and Shen-si, it flows much of the way in a canyon, and where not bounded by cliffs of rock it is shut in by walls of loess. Some portions of its channel appear to be antecedent to upwarps in the surface, and we are thus thrown back to an earlier date than that of the warping for the beginning of the river's course. But it appears from descriptions given of the valley in works of travel or by the natives, that nowhere in its lower section is the river accompanied by that wide channel of an older valley which we would expect if it had long flowed in its course across the surface. . . .

These characteristics of the river valley point to unusual youth, and apparently indicate an absence of an earlier history.

Prior to the development of the modern river, the ancient Yellow River occupied its upper course as far as the western border of Shansi. From this point it probably flowed through the present valley of the Heisen Ho into the chain of lakes which occupied the upper valleys of the Sankang Ho and the Yang Ho, and discharged through the present Yang Ho gorge, through the Hun Ho and the Pei Ho into the interior sea which has since become the Great Plain of China.

This is shown by the system of terraces of the upper Yellow River, which apparently extend from Ningsiafu in Kansu to Yenking Chow, north and west of Peking, and which occupied the valley of the chain of lakes mentioned. (See Plate I.)

That this deposit was formed in fresh water is shown by the presence of the shells found in the terrace of the Te Hai. The uniform character of the loam in the different basins, and in all parts of the same basin, its great extent, and the fineness of the material of which it consists, are conditions which prove that it is not of local origin, or derived from the detritus of the neighboring shores, but that it was brought into the lakes by one or more large rivers which must have drained an area of great extent. Now throughout the region in question, the only rivers are those of the Yang Ho and Sankang Ho basin, and, independently of the fact that these streams drain a very small area, the valley systems of these were almost entirely occupied by the lakes.

Indeed the only direction from which a river of any importance could have come, was from the west, in which case it could only have been the Hwang Ho (Yellow River). (See Raphael Pumpelly's "Geological Researches in China, Mongolia and Japan," page 42.)

The later development of the great system of normal faults which extend through the mountains of Shansi for some 450 miles in a general northeasterly and southwesterly direction, was accompanied by a general settlement toward the southern end of the fault block and a rising to the northern portion of Shansi near the Mongolian border. This movement either produced an upheaval and consequent obstruction in the eastern course of the ancient Yellow River near the western border of Shansi or accomplished the same result by opening a lower outlet channel to the south and east around the end of the fault block, and thus caused the Yellow River to radically change its point of entrance into the Great Plain of China.

Evidence shows that when the Yellow River first began to discharge its waters through the present outlet near Mengtsing the land stood at an elevation of several hundred feet below its present altitude, and the silt brought down by the river was deposited in deep water, forming a normal delta adjacent to the mountains, and at the same time with other streams building up, with lighter material, the general sea bottom where the Great Plain of China now exists. Later, in geological times, diastrophic movements raised this sea bottom to or above the water surface, thus forming the base at least of the Great Plain. This upward movement of the sea bottom and of the Yellow River delta caused the degradation of the bed and delta of that stream and others flowing into this sea, as evidenced by the terrace systems in the valleys of the Yangtse and Yellow rivers, where they debouch from the mountains, referred to by Willis and Pumpelly. As this upward movement continued, the consequent degradation probably increased the deposits in the sea, thus increasing the slope of the sea bottom from the mountains to the present Gulf of Chihli and the Yellow Sea. When this upward movement caused the Great Plain of China to emerge from the sea, the length of the Yellow River was materially increased and its action was then reversed. Instead of cutting its own bed and delta, it commenced to build them up so as to form a normal gradient for its greatly increased length. Before this could be done, man interposed and began to confine the stream by dikes.

The great disasters caused by this river in the past, which have given it the name of "China's Sorrow" and which are still a menace to the Great Plain of China, are largely the result of the

fight which has been going on for centuries between the Yellow River, which is carrying on the development of a normal delta plain under abnormal conditions, and the Chinese people, who have attempted for four thousand years or more to confine this stream within limited bounds in order that they may cultivate as much as possible of the great plain of eastern China and support thereby a comparatively dense population.

LAWS OF DELTA GROWTH.

Normal deltas independently created by river systems are in general of a fan-shape and have sufficient gradient to assure the discharge of the load of silt carried by their waters at ordinary stages into the sea. Only after such a gradient is established in the normal delta plain is the delta normally further extended. The channels are, when normal, sufficient for low water conditions, but the flood waters commonly overflow the lower delta area, and the contained silt forms natural dikes along the channel and slowly builds the adjoining plain. The gradient, being sufficient for the transportation of silt, is not increased by the silting of the channel excepting as the delta is extended and it becomes necessary to maintain the gradient as the length of the stream increases. A normal stream meanders across the flood plain, cuts and fills its concave and convex bends, and as its length increases raises its bed and occasionally makes radical relocations of its lower channel.

THE NORMAL DELTA OF THE YANGTSE.

The Yangtse has largely created its own delta, which is normal and occupies the extreme southern portion of the great plain above described and the extension of such plains among the mountains and hills to the southward and eastward to the bay of Hanchow. The Yangtse has at different times varied its course so as to occupy most of this delta plain, its course being to a considerable extent modified by its delta growth and the position of the intervening mountains.

Within historic times there has been a considerable extension of the Yangtse delta plain, and doubtless a corresponding rise in its upper delta bed, or at least in its water slope to overcome the losses of gradient caused by such extension.

The conditions of a normal delta are clearly shown in the delta

of the Yangtse. Although its channel has been diked for centuries and in spite of a considerable extension of its mouth seaward, the river has not elevated its bed above the surrounding diked lands.

DELTA GRADIENTS.

The low water elevation of the Yangtse at Hankow, 600 miles from the sea, is about 133 feet above mean sea level, giving it a low water slope of approximately .22 foot per mile. The silt carried by the water, while considerable, is small as compared with the silt content of the Yellow River. The normal delta gradient of any stream is necessarily a function of the quantity and quality of the silt carried by the stream and of the nature of the bed over which the stream flows. In the Yellow River not only is the silt carried excessive in quantity, but it is pervious in character and is deposited on a pervious substratum that permits extensive losses in the river volume by seepage and hence produces an unusual cause for silt deposition. The Yellow River, in its effort to build up a normal delta plain, has already raised its bed at the bridge of the Hankow-Peking Railway, about 400 miles from its mouth, to an elevation above mean sea level of about 400 feet, giving it a gradient of about one foot per mile. Even this gradient, apparently excessive when compared with the gradient of the Yangtse, is totally insufficient for the normal purposes of silt transportation to the sea. A river flowing over a plain having insufficient or only sufficient gradient for silt transportation purposes has no general tendency to deepen its bed. The Yellow River, in accordance with this law, has never excavated a channel in the Great Plain of China, but has meandered over the plain in its effort to create normal delta conditions and has formed channels only by the building up of flood plains adjacent to its lines of flow.

THE CONDITION OF UNRESTRICTED FLOODS OF THE YELLOW RIVER.

The results from the unrestrained flow of the Yellow River may be understood and appreciated by its action during the times it has escaped from its diked captivity and wandered normally over the plains of China. About 1853, the river broke away from the bed that it occupied for over five hundred years, at a point about thirty miles northeast of Kaifengfu, and wandered unrestrained for many years to the northward of its former course, capturing the

bed of the Ta-ch'ing Ho and moving its mouth some 250 miles to the north of its former outlet and to the region to the north of the Shantung Mountains. About 1867 a portion of the area occupied by the unrestrained river was visited by Messrs. Ney Elias and H. G. Hollingworth.

Describing their impression of the Yellow River at the Nanshan, a small limestone hill forming the western extremity of the Tsinan highlands, Mr. Elias says:

The river at this point has no defined bed, but flows over a belt of country some ten or twelve miles in width, having merely the appearance of a flat, level district in a state of inundation; patches of ground, trees and even villages cropping up here and there; the Grand Canal traversing it in a general northwest direction until it reaches the northermost channel of the river at Palimiso, some fifteen miles west from the Nanshan. Along this fifteen miles of the canal the banks have been carried away in a number of places by the Yellow River breaking across them. The gaps are sometimes half a mile or more wide, and the current rushing through them almost obliterates the course of the canal, and renders the navigation upon it difficult. For dreariness and desolation no scene can exceed that which the Yellow River presents here. Everything natural and artificial is at the mercy of the muddy dun-colored waters, as they sweep on their course to the sea; a flood not likely to subside and a doubly mischievous one from the fact of its ever moving onwards with a swift current.

The work of the Yellow River, due to one of its breaks through the artificial dikes by which its course has been temporarily restrained, is described by Tyler as follows (see notes on the Hwang Ho or Yellow River, by W. F. Tyler):

Five years ago (in 1898) a breach occurred at Wang-chia-li-hang, some 40 li below Chi-nan, on the south bank, which devastated a large area of country and caused the terrible distress which resulted in the organization of relief work by foreigners. The effect of this flood I was able partly to examine while going down the Hsiao-Ch'ing-ho, and afterwards by riding along the course of the torrent from where the breach occurred, I estimated that at least 100 square miles were turned into a sandy waste of almost absolute sterility, while over an area of 200 square miles a deposit of more or less sterility was laid. . . . At the few places where actual examination was possible the depth (of the deposit) was from 6 to 10 feet, and the statement of the natives was that it varied from 4 to 8 feet. The river Taotai (official in charge) concerned considers the average depth to have been about 5 feet. Now, if we adopt the depth of 5 feet, it is seen that the sand which was poured out of the river in a single flood was about one-fifth cubic mile (28,000,000,000 cubic feet)!

Describing the last great change in the course of the Yellow River, Tyler says:

In 1853 the Hwang Ho broke its dike near K'ai-feng, and about 270 li to eastward of the breach the defluent waters found the bed of the Ta-ch'ing-ho, as if it had been specially prepared to conduct the Yellow waters for the remaining distance, of about 800 li to the sea. The bed of the Ta-ch'ing-ho was at that time about 1 li wide, and was sunk in the plain to a depth of some 50 feet or so. This bed, to commence with, was more than sufficient to hold even the flood waters of the river; but the silting up must have commenced at once, and each year the flood waters must have come closer and closer to the top of the river bank.

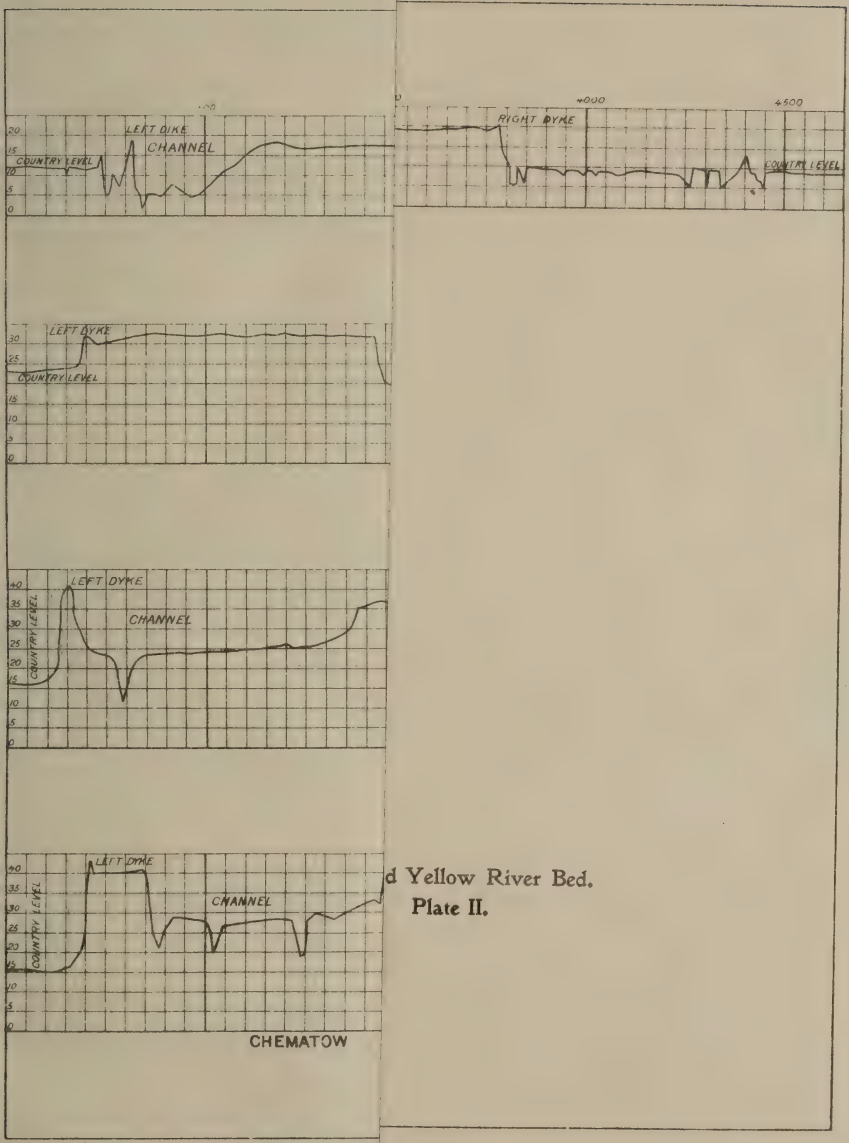
After a period of 30 years, that is, in 1883, the flood waters began to overflow and gently flood the plain. In consequence of this, the Governor of Shantung constructed dikes on both sides of the river from Tung-s-hsien to the sea. These dikes are the present existing outer dikes of the river. Previous to this the dikes had gradually been continued down from Honan to Tung-s-hsien. . . .

In the meantime the Hwang Ho gradually raised the plain between its embankments, until now (1903) the flood level is about 15 Chinese feet above the level of the original plain—that is to say, the flood level has risen 15 feet in 20 years, that is, three-quarters foot a year.

Historical evidence indicates that the Yellow River has been diked for about four thousand years and has in no case ever created on any course a gradient sufficient for normal silt transportation. This diking was therefore begun under conditions radically different from those of rivers flowing through deltas completed normally by their own silt deposits. On account of the necessities of a thickly populated country, the wandering branches of the Yellow River flowing unrestrained across the flat plain of China were early confined to a more narrow range by an extensive dike system. These dikes were built locally and insecurely at first, but were gradually strengthened, systematized and improved until they finally developed into an extensive dike system under the somewhat experienced and systematic care of the local and national authorities. In its confined channel, between primitive dikes, the Yellow River has been building up a restricted delta and endeavoring to create a gradient commensurate with its needs. This has involved the raising of both its bed and the limited flood plain within the restricted dikes until they are frequently high above the surrounding level of the great Chinese plain (see Plates II* and III*). As

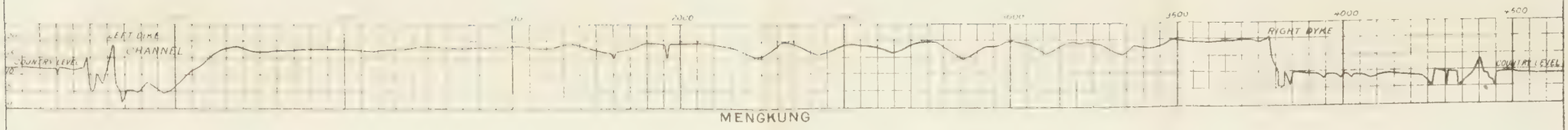
*Scales of these plates are expressed in English feet.

the heights of its bed and flood plain increased within the restricted area the river frequently broke through its confining dikes, and while sometimes recaptured and returned to its artificial limits, often permanently threw off its bonds, devastated the country and wandered uncontrolled over a new portion of the plain, carrying on its normal work of delta building in a wide track until again confined in a new and restricted course to enable the Chinese people to reclaim temporarily the lands necessary for their sustenance.

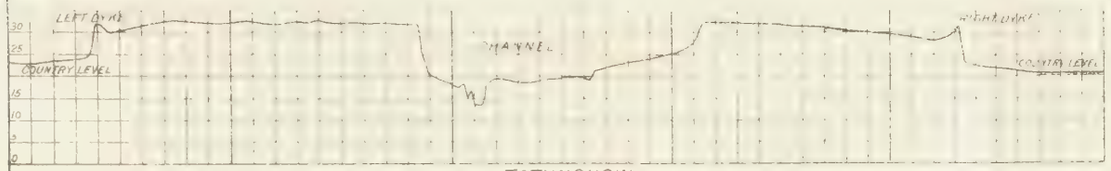


d Yellow River Bed.

Plate II.



MENGKUNG



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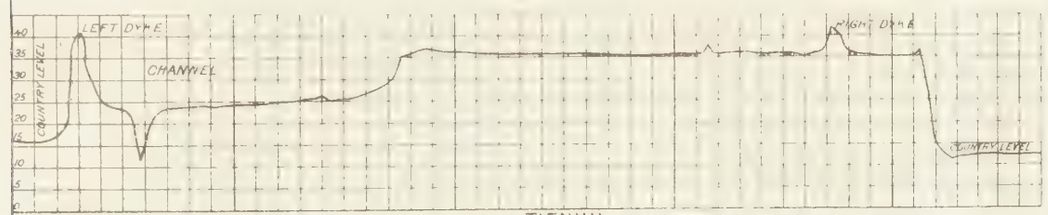

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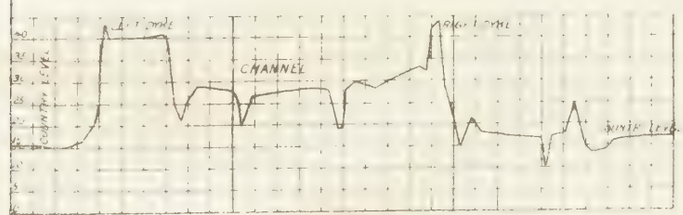
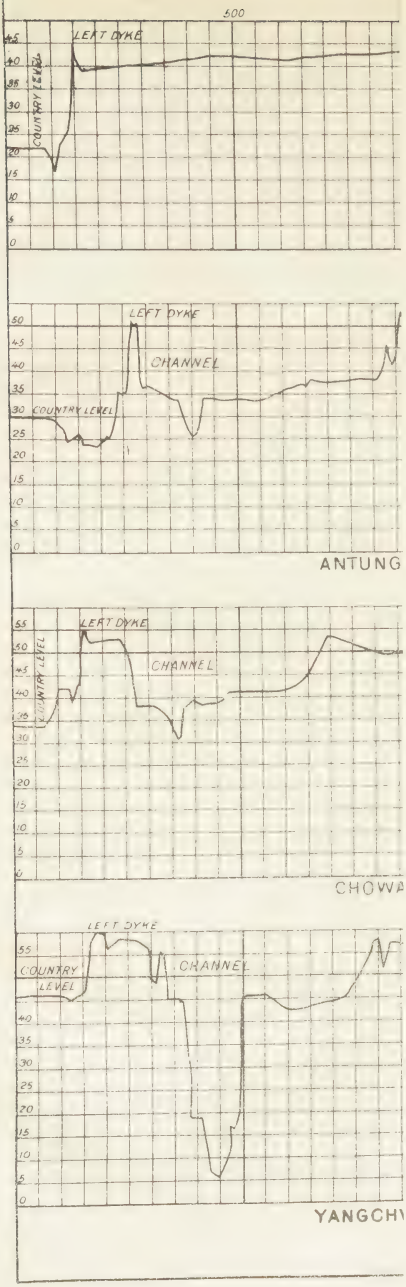


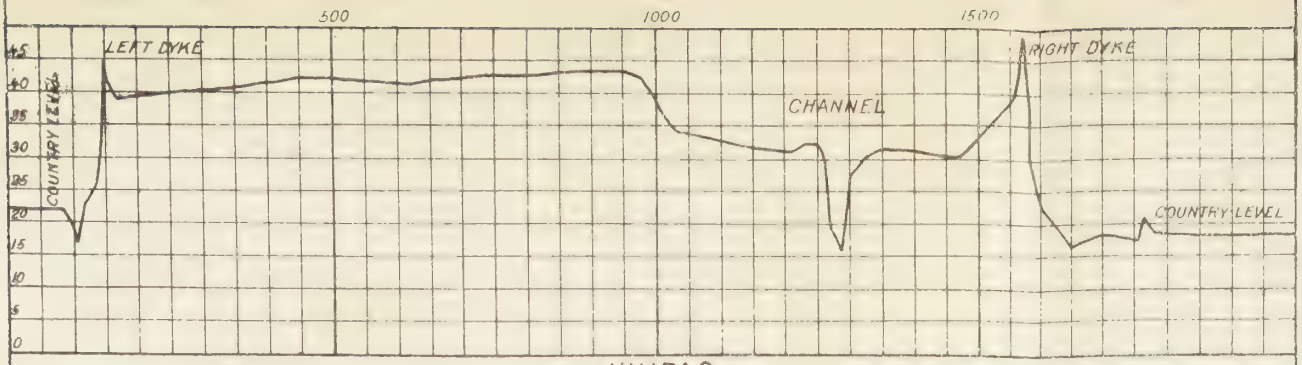
CHEMATOW

Cross Section of Old Yellow River Bed.
Plate II.

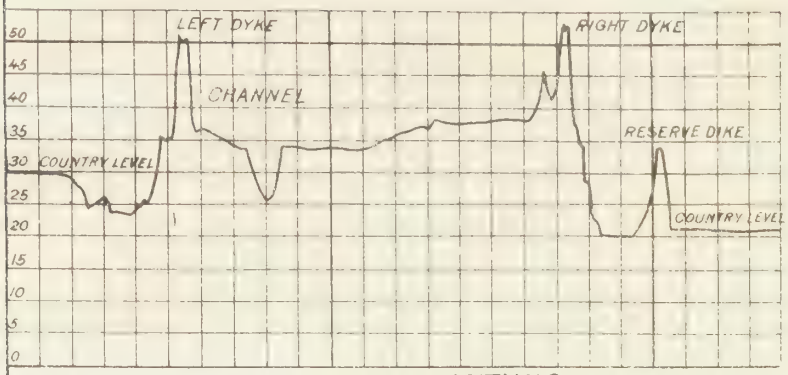


Cross Section of Old Yellow River Bed.

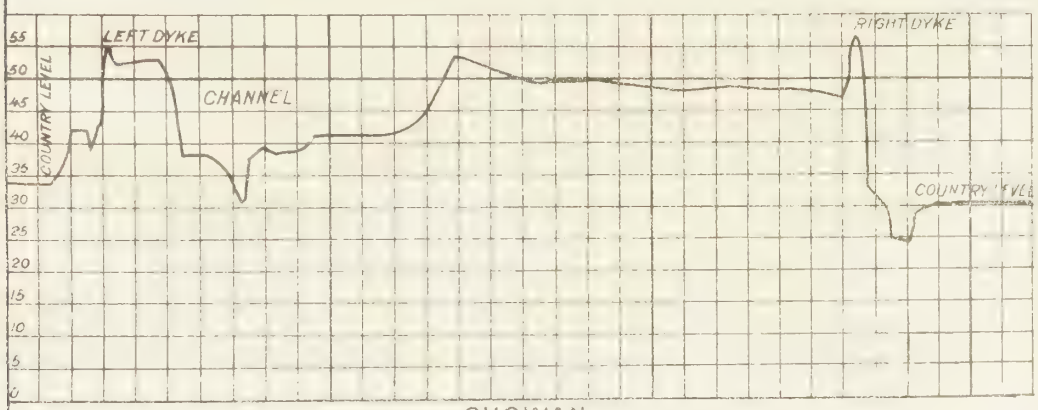
Plate III.



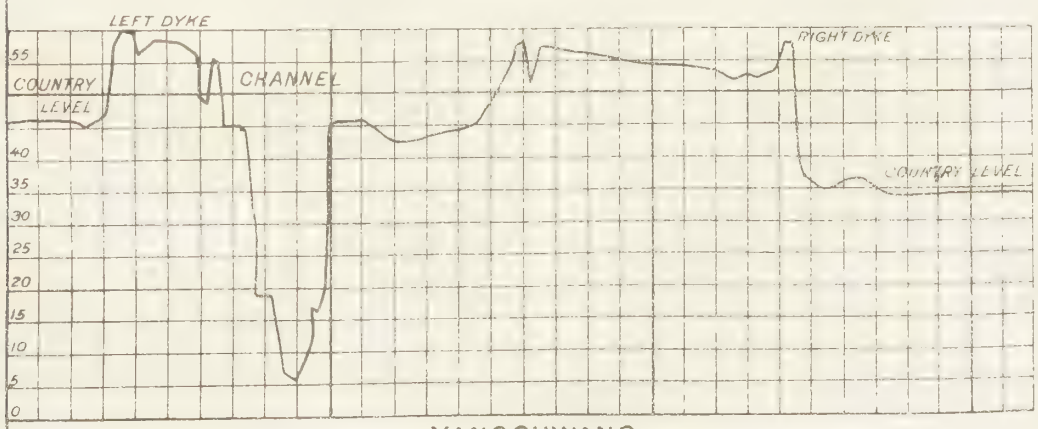
KIUPAO



ANTUNG



CHOWAN



YANGCHWANG

Cross Section of Old Yellow River Bed.

Plate III.

Draft of Vessels Entering and Leaving New York Harbor, January 1 to June 30, 1914.

BY

Maj. E. J. DENT,

Corps of Engineers; Assoc. M. Am. Soc. C. E.

The following classes of vessels entering or leaving New York Harbor are required to pay pilotage:

(a) All foreign vessels;

(b) All vessels from foreign ports;

(c) All vessels sailing under registry to or from New York.

Other vessels may employ pilots and thus pay pilotage, but they seldom do so. The masters and mates of coastwise vessels are licensed pilots in nearly all cases and no pilotage is paid by such vessels.

The charge for pilotage is based in part upon the draft of the vessel at the time of entry or departure, and a record of such drafts is kept by the Sandy Hook pilot offices. A compilation of this data for the period January 1 to June 30, 1914, has been made. The total number of pilotage fees paid during that period was 4,576. The result of this compilation is shown in the following table.

The observed "squat" of some of the largest vessels leaving this harbor has occasionally exceeded 4 feet.

Drafts between	Total N.Y.H.	Accum. figures	Per cent
38 and 39 feet	3	3	.0655
37 and 38 feet	5	8	.1749
36 and 37 feet	7	15	.3277
35 and 36 feet	13	28	.6118
34 and 35 feet	7	35	.765
33 and 34 feet	18	53	1.158
32 and 33 feet	41	94	2.054
31 and 32 feet	41	135	2.95
30 and 31 feet	39	174	3.802
29 and 30 feet	68	242	5.302
28 and 29 feet	167	409	8.937
27 and 28 feet	133	542	11.844
26 and 27 feet	169	711	15.537
25 and 26 feet	231	942	20.585
24 and 25 feet	239	1181	25.808
23 and 24 feet	287	1468	32.08
22 and 23 feet	353	1821	39.794
21 and 22 feet	340	2161	47.224
20 and 21 feet	295	2456	53.671
19 and 20 feet	318	2774	60.62
18 and 19 feet	335	3109	67.941
17 and 18 feet	268	3377	73.798
16 and 17 feet	177	3554	77.666
15 and 16 feet	211	3765	82.27
14 and 15 feet	234	3999	87.39
13 and 14 feet	182	4181	91.368
12 and 13 feet	164	4345	94.951
11 and 12 feet	123	4468	97.639
10 and 11 feet	92	4560	99.65
9 and 10 feet	14	4574	99.956
8 and 9 feet	2	4576	100.
Total	4,576		

Editorial Note.

As announced in No. 25 of the PROFESSIONAL MEMOIRS, awards of cash prizes have been made to the authors of the four best articles that appeared in Nos. 25 to 30, inclusive. The conditions of eligibility for consideration for these awards were the same as those announced for a similar distribution of prizes for articles appearing in Nos. 31 to 36, inclusive: the offer is open to all subscribers except officers of the Corps of Engineers with more than 10 years' service in the Corps.

The awards are as follows:

First Prize, $50. To Mr. Nicholls W. Bowden, Junior Engineer, U. S. E. D., for the article "Improving Tennessee River above Chattanooga, Tenn., by Regulation or Open-channel Method," published in No. 30.

Second Prize, $25. To Capt. E. L. Daley, Corps of Engineers, for his article entitled "The Modern Siege," which appeared in No. 26.

Third Prize, $15. To Capt. Lewis H. Watkins, Corps of Engineers, for the article entitled "Map Making in Connection with Land Defense for Seacoast Fortifications and Seacoast Cities," published in No. 28.

Fourth Prize, $10. To Mr. C. W. Durham, Principal Assistant Engineer, U. S. E. D., for his article "Winter Work in the Construction and Repair of Dams and Shore Protections in the United States Improvement of the Upper Mississippi River," published in No. 29.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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COL. JOHN JAMES ABERT
CADET U. S. M. A., 1807-1811.
BORN 1788—DIED 1863.

SEE PAGE 204

Procedure in the Investigation of a Proposed Water Power Development.

BY

Lieut. EARL J. ATKISSON,
Corps of Engineers, U. S. A.

In the study of any waterway or system of waterways consideration is given to one or more uses of the stream, its water or other features, depending on the interest which prompted the study. Until recently only one of the more important uses of waterways, that of navigation, has received any systematic treatment. Federal control has been exercised in the interests of navigation, with a result that a department has been developed which can properly and correctly investigate and report on all questions affecting navigation. However, another important and legitimate use for certain parts of waterways, the development of water power, has received and is receiving such consideration as will eventually insure its economic development in keeping with the best interests of the country.

The Congress has plainly indicated its pleasure regarding water power possibilities in navigable streams, by requiring an investigation and report on the same in connection with investigations carried on in the interests of navigation. Congress has rightly assumed that these two interests can be properly investigated by one department, since they are not antagonistic but inter-related and cooperative. An appreciation of the duty involved in this trust is not possible without a clear understanding of the many elements which go to make up a water power development.

That water power development is desirable is not a question, but whether a particular one is feasible and in accord with the best interests of the people to be served is of sufficient importance to warrant an investigation of all available data. Where the results of original investigations attract private capital to an undertaking at an earlier date than would otherwise have been the case, or where the same acts as a basis of an economical and honest development.

the public has been served. Likewise in granting permits authorizing developments based on private or public investigations, an intimate knowledge of all conditions is necessary to insure that the development at the point or points in question is a maximum development, or the best economic development.

A water power development has come to mean the conversion of the energy of falling water at one or more points by means of suitable apparatus into electrical energy with the necessary transmission and distribution systems. The interested views of the promoter would frequently lead one to believe that the inherent value of the falling water constituted the major portion of the total value of a development. This is far from true. While the energy of the falling water takes the place of the potential energy of the coal, which is the principal expense in the operation of a steam plant, there are other charges, such as the interest on the larger investment, that frequently more than counterbalance this initial gain.

Unless a particular development is within the limit of the minimum flow, it must have storage or other plants, either water or steam or both, to fall back upon, all of which involves larger investment and additional expense. Water power is inferior to steam power in that it is subject to climatic and seasonal effects, more frequent interruptions and unsteadiness of pressure, and only when reduced costs outweigh these disadvantages may a water power development be considered feasible.

A procedure in the investigation of a proposed water power development which will furnish sufficient information to allow a definite decision being rendered regarding its value should cover the following essential features:

1. Drainage area, rainfall and run-off;
2. Selection of site;
3. Hydraulic head;
4. Power;
5. Plant design;
6. Cost;
7. Report.

With the progress which has been made in obtaining and recording rainfall, run-off and stream flow data, together with past experience, a proposed water power development may be approached with quite the same certainty of obtaining a satisfactory solution, as exists in the selection of a site for a lock and dam, its design and estimate of cost together with the estimate of its satisfactory operation and benefit to navigation.

I. DRAINAGE AREA, RAINFALL AND RUN-OFF.

Before proceeding to the detailed investigation of a project, certain general information regarding the stream flow and head will be known. These two factors affect directly the amount of power available and both being variable, naturally produce a variable power characteristic. It, therefore, becomes necessary to know the regimen of the stream before determining the power.

A large amount of information bearing on stream flow is now available, and all such information bearing on the locality in question should be obtained. These sources of information are as follows:

U. S. Geological Survey, Water Supply Papers, Annual Reports and Bulletins;
U. S. Weather Bureau, Annual and Monthly Reports;
U. S. Engineer Department, Reports;
Reports of State Engineers, State Geological Surveys, Water Supply Commissions, etc.

Where records are available for a long term of years, they may give the regimen of the stream with sufficient accuracy to warrant proceeding with the investigation. Such a condition is unusual, and it ordinarily becomes necessary to make a study of the factors affecting stream flow to verify or supplement such records as are available.

The stream flow comes directly or indirectly from the rainfall on the drainage basin of the stream. The indirect source lies in the ground water which reaches the stream through springs and seepage. The uniformity of this supply makes it the chief source of a stream available for water power. In humid parts of this country, the ground water has been estimated as some 25 per cent of the volume of the first hundred feet of sub-soil, and equivalent to six or seven years rainfall. Any tendency acting to reduce the ground reserve should be investigated and due allowance made in the estimate of stream flow both for the present and future.

The stream flow is affected directly by the run-off from the drainage basin. The percentage of run-off is affected by all of the following to a greater or less extent:

1. Amount of rainfall;
2. Rate of falling;
3. Condition of surface;
4. Underlying geological strata;
5. Temperature;
6. Direction and velocity of wind;
7. Vegetation;
8. Surface topography;
9. Evaporation.

The number of variables involved makes drawing any general conclusions as to the ratio of run-off to rainfall out of the question.

However, without attempting to deduce an equation or formula which will not apply to a special locality let alone the general case, it is necessary to make a study of all rainfall and run-off records, together with the characteristics of the drainage basin. This study should not be restricted to the one locality under consideration but should extend to adjacent similar drainage areas, especially when more or better records are available.

After obtaining all available data and completing the study of conditions, the judgment must be exercised in giving the proper weight to the average annual flow as well as the minimum and maximum flows which are to be considered later in the design of works and in the power estimate. A decision should not be rendered until detailed records for rainfall, run-off and stream flow have been obtained for at least one year and then only in exceptional cases. This condition applies with equal force to preliminary investigations and may necessitate the establishment of rainfall and gauging stations to obtain the necessary data.

Table I shows a typical rainfall-runoff record. It will be noted that the run-off for both March and April exceeded the rainfall, also that during the months of heaviest rainfall, July and August, the run-off was a minimum. Should the records over a number of years correspond closely with this and be borne out by the characteristics of the drainage area, no great trouble should be experienced in making a logical estimate of the effect of run-off on stream flow.

TABLE I.—*Susquehanna River above Harrisburg. Drainage Area, 24,030 Square Miles.*

	Rainfall.	Run-off.	Per cent run-off.
October	3.02	0.90	30
November	2.63	1.08	41
December	2.97	1.75	59
January	2.73	1.94	71
February	2.67	1.98	74
March	3.35	4.48	134
April	2.76	3.43	117
May	3.96	2.07	41
June	3.98	1.25	24
July	4.11	0.83	20
August	4.16	0.77	15
September	3.04	0.61	25
Year	39.38	21.09	53

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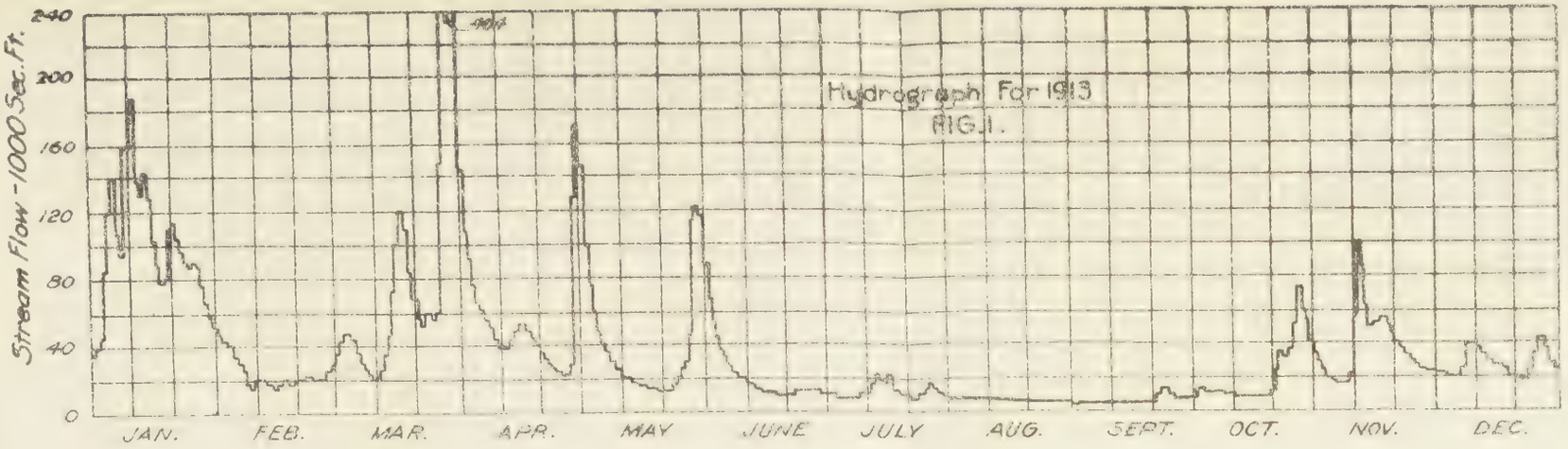
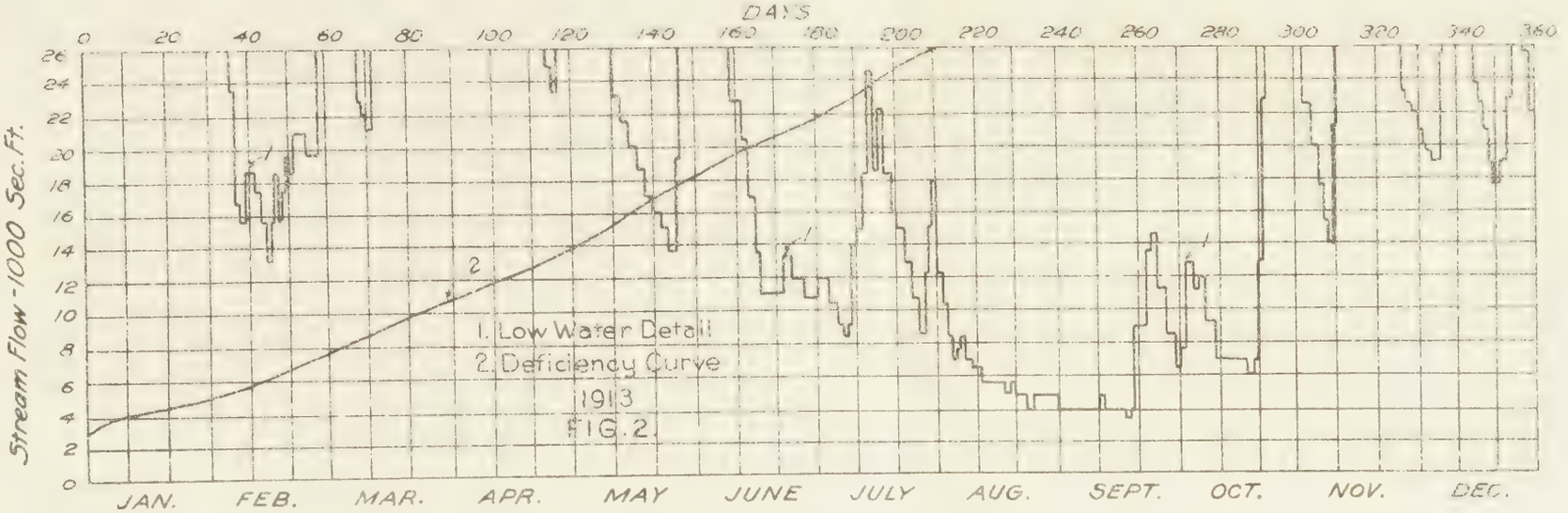
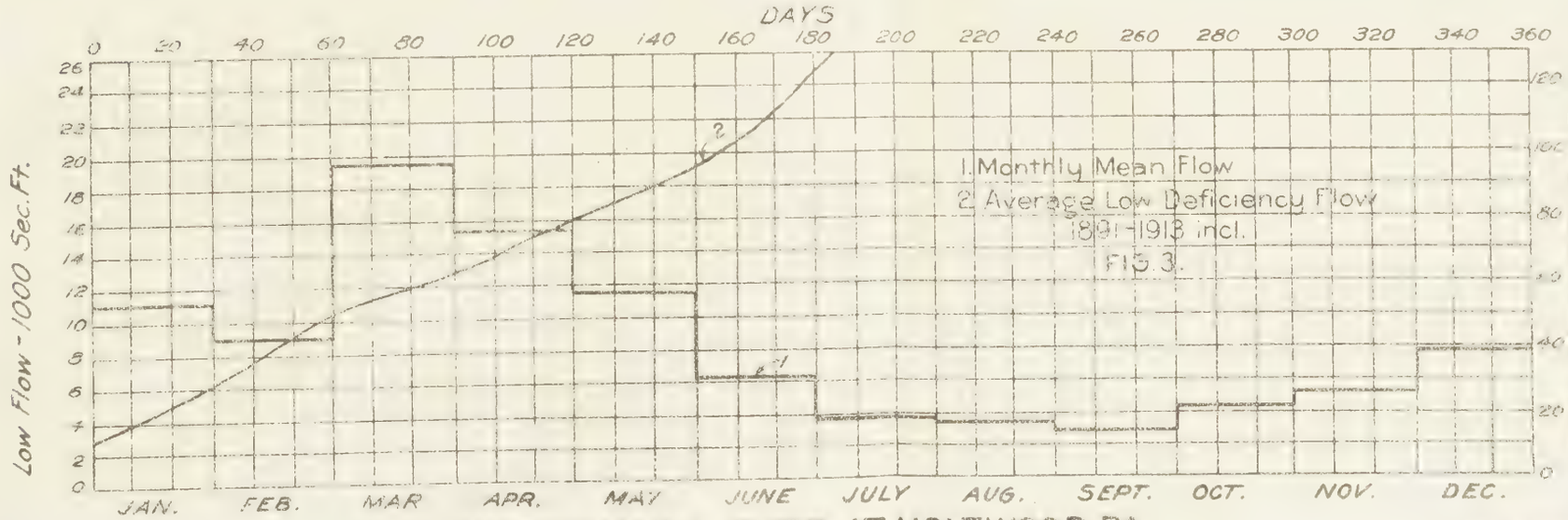


Hydrograph For 1913
FIG. 1.



1. Low Water Detail
2. Deficiency Curve
1913
FIG. 2.



1. Monthly Mean Flow
2. Average Low Deficiency Flow
1891-1913 incl.
FIG. 3.

SUSQUEHANNA RIVER AT HOLTWOOD, PA.

PLATE I.

Where stream flow records are available they are usually for particular points, not necessarily those at which a development is contemplated. After obtaining the area of the drainage basin, either from existing maps or surveys, proper allowance can be made for the difference existing between the area for which records are available and that for the location considered. The data should then be corrected and put in such form as to make it available for use.

This consists in first preparing the customary hydrographs either in the form of tables or curves. Figures 1 to 3 (Plate I)

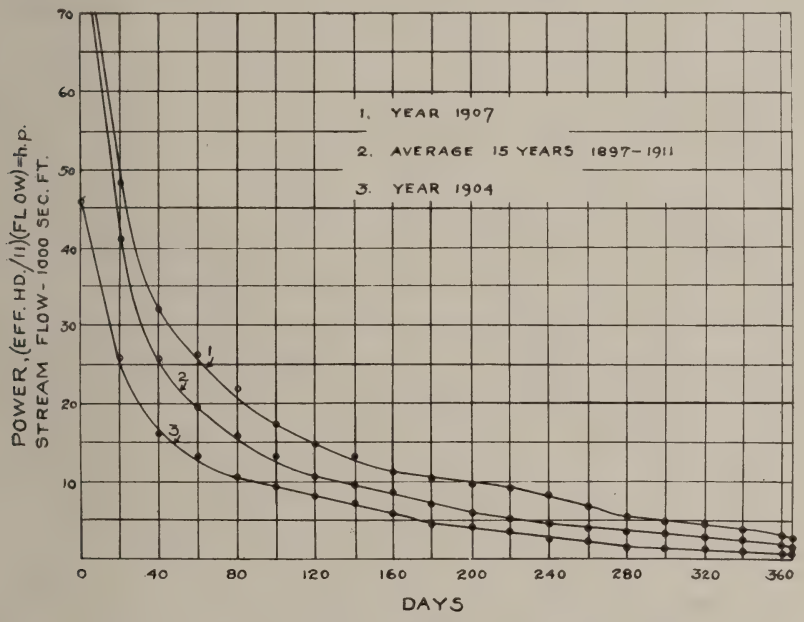


Fig. 4. Duration Curves, Aqueduct Dam, Potomac River.

show data kindly furnished by Mr. H. W. Lowy, Hydraulic Engineer, Pennsylvania Water and Power Co., for the Susquehanna River at Holtwood, Pa. Curve 1 on Fig. 1 shows a typical one year hydrograph, the stream flow in second feet being plotted against time. Curve 1 on Plate I, Fig. 2, shows the low water detail of the hydrograph of Fig. 1. The ordinate scale is increased, and all values of stream flow are neglected above a certain value approximating the maximum or other value which is considered as available for power. Curve 1 on Plate I, Fig. 3, shows the monthly mean flow for the years 1891 to 1913, inclusive.

These curves readily show maximum and minimum flows and seasonal variations, but are not in themselves in a form suitable for power investigation. This leads to the development of curves showing either deficiency in flow or duration of flow. The former may be known as deficiency curves and are plotted, using as abscissæ the number of days during a year in which the flow, indicated as ordinates, was not equalled. Thus, Curve No. 2, Fig. 2, shows such a curve for the year 1913, at Holtwood, Pa. Curve No. 2, Fig. 3, shows the average low flow for the period 1891 to 1913, inclusive.

Curves showing duration of flow may be known as duration curves and are plotted, using as abscissæ the number of days during a year in which the flow was at least equal to that indicated by the ordinates.

Figure 4 shows three such curves representing the limiting and average values of the stream flow at Aqueduct Dam near Washington, D. C., on the Potomac River, plotted from data included in the report on the Great Falls water power project, 1913.

II. SELECTION OF SITE.

Ordinarily, the approximate location of the power house, head and tail races, and dam is indicated by falls or rapids in the stream. The exact location should, however, receive attention on the following grounds:

(a) Relation of different sites on same stream;
(b) Character of works necessary;
(c) Foundations;
(d) Methods of construction and supply;
(e) Property rights.

(a) Where the principal fall in a stream occurs at one place or in a relatively short stretch, usually only one solution is possible and that apparent. When the fall occurs at different places either abruptly or in the form of rapids, several solutions may be made. By building one high dam, the head may be made available at one place. Again the cost of the dam, or flowage rights or both, may prohibit this, and necessitate developing only part of the fall by a lower dam, or of developing the total fall at two or more places. The problem is not to develop the maximum head available but to make the most economic development. If a development already exists on the stream, effort should be made to determine reasons why its site was chosen in preference to those under consideration.

(b) Advantage should be taken of all natural features of the ground. Frequently a section crossing islands may cut down the length of dam required. Again, an island may be used in obtaining a natural head race or tail race. An example of the latter is well illustrated in the development of the Pennsylvania Water and Power Co. at Holtwood, Pa.

Figure 5 shows the location of this plant, and how a natural tail race extending downstream two-thirds of a mile and giving a marked increase in effective head was obtained by the construction of

Fig. 5.

a short wall between the power house and head of Piny Island. A low diversion dam may be built at the head of a reach, and the water led in penstocks to the power house situated at a lower level.

In approaching this phase of a development, it is, of course, desirable to refer to several more or less typical layouts of plants already in satisfactory operation, but it must be remembered that none can be followed in detail. Each new water power development is a new problem in design, and only by so considering it can the best be obtained from existing local conditions.

(c) The impounding of water which usually accompanies a water power development involves a risk which must be met by safe, substantial construction. The investigations carried on to determine the character of the foundations at the site selected must be in keeping with best practice. Where suitable foundations are not found or their preparation would be unduly expensive, a new location of works must be made or the expense justified by gain in power or saving in other construction costs.

(d) The location of the water power development is usually such as to make difficult the supply of material, construction plant and later apparatus and supplies. In the selection of a site, care must be taken to see that the necessary construction and supply is economically possible. It is emphasized that the water power development is rarely practical unless it delivers power to the consumer at a price *below* that of existing steam plants.

(e) The property rights covering the location of the plant and accessories, right of way for transmission lines, and flowage rights for pondage and storage must be purchased. In the private enterprise, in the absence of the right of eminent domain, the exorbitant prices charged for property may prohibit the development.

III. HYDRAULIC HEAD.

Having settled upon the possible location of the development, the actual fall available should be determined with a spirit level. This head is usually made available by the construction of some type of dam. The actual design of the dam should be in accordance with best practise for this type of structure, having due regard for maximum flood conditions and also the service of the turbines. Its height, however, must be investigated on the following grounds:

(a) Overflow of property;
(b) Interference with existing power rights above and below site;
(c) Interference with other vested or public rights;
(d) Cost of structure.

(a) Reference has already been made to the purchase of flowage rights. Upon the selection of a certain height of dam, the back water area may be determined for the water level corresponding to this height. Only during seasons of low stream flow, when the turbines take the greater part of it, will this area give an indication of the extent of overflow. Discharge curves for a

number of cross sections of the stream included in the probable back water area must be made. These discharge curves are obtained by plotting gauge heights against discharge, as shown in Fig. 6.

Using the hydrographs for the stream and the discharge curves for the dam and different sections of the back water area, the extent of this area may be obtained for all values of stream flow, including that of flood. Knowing the extent of the area to be flooded, an investigation should be made to determine whether the property can be purchased or flowage rights obtained at figures which will allow going ahead with the project. Different heights of dam, while they may not affect the power output to any great extent, may have a marked effect on the extent of the back water area, and should be investigated.

(b) Where the back water extends upstream such a distance

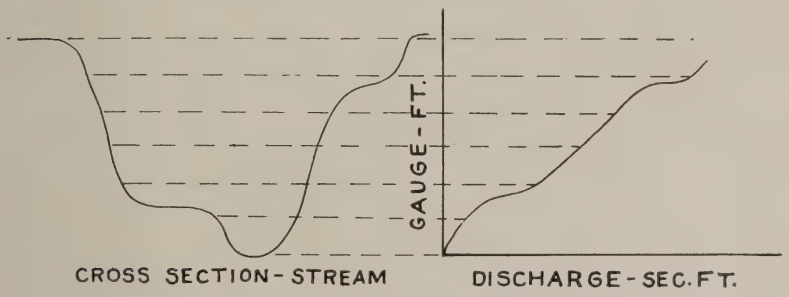


Fig. 6. Discharge Curve.

as to cut down the effective head or otherwise interfere with existing developments above, settlement must be made or the power rights purchased; in like manner where the water is held back in storage to the detriment of installations situated below.

(c) Highway and railroad bridges, roads, and other private and public properties damaged or destroyed must be provided for, or height of dam corrected accordingly.

(d) Where other conditions do not place a limit on the height of the dam, the only limit may be taken as the cost of the structure. This should be in keeping with the power obtained.

Provision may be made for some type of movable dam or system of flashboards for the spillway of the permanent dam, for use during seasons of low stream flow. Provision should be made to insure taking care of sudden increases in stream flow, so as not to risk the safety of dam or subject development to flood damages.

With the height of dam fixed, it becomes necessary to determine the effective head corresponding to different values of stream flow. In the usual case the reduction in head during floods is appreciable, the head being a maximum with minimum flow. Ordinarily, the turbines are designed for best operation under low water conditions, and the reduced head during flood seasons may so reduce the power output as to necessitate installation of additional units.

Referring to Fig. 7 with rating curves for the sections of the stream corresponding to the inlet to the head race and outlet of the tail race and with staff gauges referred to the same datum, the difference in readings corresponding to the same discharge will give the effective head for that discharge. In this manner an

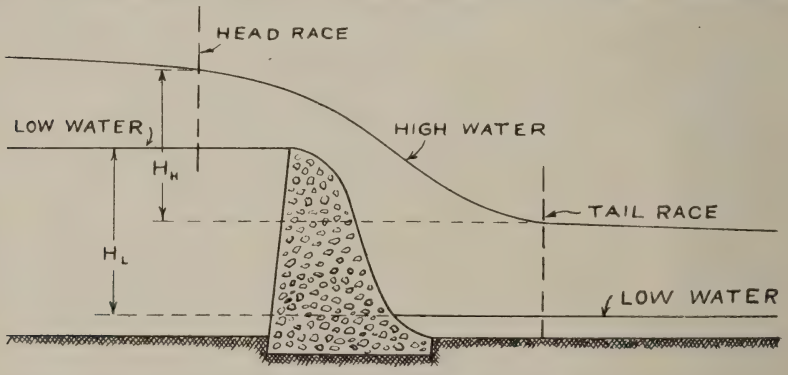


Fig. 7. Variation in Effective Head.

"effective head" chart may be constructed to correspond to any hydrograph.

Figure 8 shows such a curve for the proposed Great Falls development on the Potomac.

In the design of the plant care must be taken to make the head race of such ample dimensions that all turbines may be supplied without drawing down the water surface; and likewise the tail race, to prevent the water backing up. Where penstocks or conduits are used, friction losses must be balanced against cost of increased size.

IV. POWER.

Using the duration curves already obtained together with the "effective head" chart, corresponding power curves are readily constructed. Using the deficiency curves, corresponding defi-

ciency power curves may be drawn. The horsepower available at generator terminals is expressed as follows:

$$\text{h. p.} = \frac{(62.5)\ (\text{Flow in sec. ft.})\ (\text{Effective head})\ (\text{Eff. turbine})\ (\text{Eff. generator})}{550}$$

Assuming a turbine efficiency of 87 per cent and a generator efficiency of 92 per cent, the expression becomes:

$$\text{h. p.} = \frac{(\text{Flow in second feet})\ (\text{Effective head})}{11}$$

Where the effective head remains constant, the ordinate scale

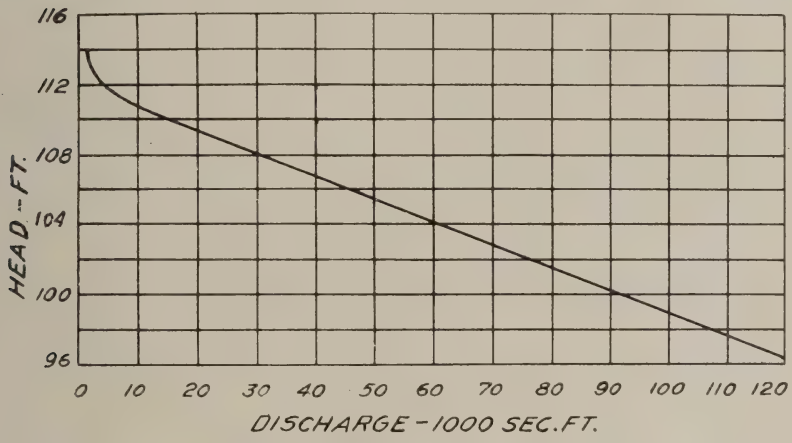


Fig. 8. Effective Head Chart.

of the duration curves need be multiplied by the constant, $\frac{\text{Effective head}}{11}$, to give the corresponding power curves. Where the effective head does not remain constant, the necessary changes must be made, either by using a variable ordinate scale or constructing new curves.

The lowest point of Curve 3, Fig. 4, gives the value of power which the development can furnish at all times without regard to pondage or storage or auxiliary power. This is considerably below the lowest value of Curve 2, Fig. 4, which shows average conditions. It is readily seen that with a plant designed to use only the minimum recorded flow, there would be a waste of power on every day except the day of minimum flow and only under the

very best of conditions, such as exist at Niagara Falls, would the power available make the development practical.

In estimating the amount of power in a stream whose development may be considered best or most reasonable, different engineers have assumed different bases depending upon the stream flow at certain times or for certain periods. Table II*, compiled by Dr. Cary T. Hutchinson and published in March, 1914, *Proceedings of the American Institute of Electrical Engineers,* shows different bases of flow applied to the Susquehanna River at Holtwood, Pa., for twenty years, 1891-1910. The relative flow figures, which are a measure of the power, show wide variations and prove conclusively the impossibility of applying an arbitrary rule to stream flow data to obtain a value for which a development shall be made.

TABLE II.

Basis of flow.	Flow second feet.	Relative flow.
1. Lowest day in the period	2,200	1.00
2. Average for the lowest week in the period	2,800	1.20
3. Average of the two lowest 7-day periods in each year for 7 years (1904-1910)	6,300	2.87
4. Average flow for the lowest week of the lowest month of the six highest months for each year of the period (1904-1910)	20,000	9.10
5. Average of annual one-day minima	5,200	2.37
6. Nine-months flow for lowest year	5,400	2.45
7. Six-months flow for lowest year	12,000	5.48
8. Nine-months flow for average year	12,000	5.48
9. Six-months flow for average year	25,000	11.40

Stream flow data alone can not furnish a satisfactory basis upon which to rest the extent of a development, but a proper determination can be made when combined with a study of:

(a) Pondage and storage possibilities;
(b) Existing and future power markets;
(c) Available auxiliary power.

(a) Pondage implies a limited reservoir capacity, only sufficient to permit using the daily inflow at any rate of outflow through the turbines in keeping with the load variations. Pondage allows daily regulation, but does not extend to weekly or sea-

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sonal regulation. In the absence of pondage, the maximum load must not exceed that obtained with the minimum flow unless auxiliary power in some form is provided. Fig. 9 shows a typical daily load curve. Without pondage, letting A-B represent the power available from the flow of the stream for the *day,* it would be manifestly impossible to carry the peaks of the load. With pondage, however, the entire load could be carried, since the average of the load, indicated by C-D, falls below A-B.

Storage differs from pondage only in amount. Its effect is to raise the minimum power of a stream by tiding the discharge over seasons of low water supply. The greater the storage, the

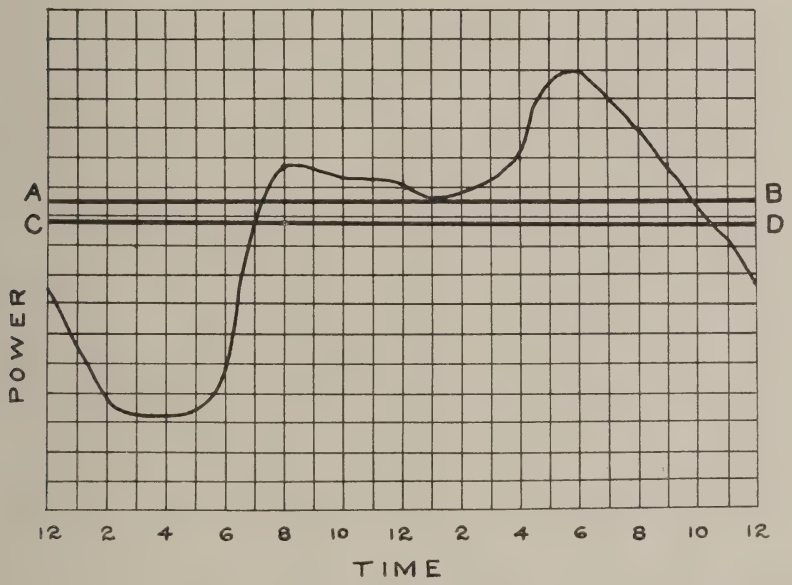


Fig. 9. Load Curve.

higher the power output becomes, until the average power of the stream is reached. An investigation of the drainage area should be made with a view to the development of storage. Where storage possibilities exist, their amount should be computed and their effect on power output determined. This output when applied to the power deficiency of the stream will give a direct measure of the increase in capacity of the stream, which allows a determination of the amount that may be spent in the purchase of sites and the construction of reservoirs.

The type of development will have much to do with the development of storage. With a low head, a relatively large amount of

water is required for a given amount of power, necessitating large storage capacity to obtain any appreciable gain. Unfortunately, low head developments are usually found in localities where large storage capacity is difficult to obtain or out of the question. In high head plants the reverse is the case, and usually large reservoirs can be built at reasonable cost.

(b) It is evident that a market for power must exist or be possible of creation before any development can be considered favorably. An investigation of the power market should consist first, of a census of power used in the locality, and second, probable future requirements.

The tendency of recent years toward the development of central stations, together with marked increases in the efficiency of large steam driven units, has resulted in the production of such satisfactory electrical power at low cost as to warrant its universal use. In commercial districts, the demand crowds the supply. In such cases it is only reasonable to expect that available water power will be developed with a view to combining it with existing steam plants. Such a condition is most favorable in that it consists of a developed and growing market and is provided with sufficient auxiliary power to take the peak loads and provide a necessary reserve. Under such conditions, in which power requirements are definitely known and future requirements can be predicted with reasonable certainty, the problem becomes one of balancing the value of the power required against the cost of the water power development or an addition to the steam plant.

In investigating existing load conditions it should not be done with the attitude that the proposed water power development will be able to obtain a sufficiently large proportion of the existing load to force other sources out of business. Localities in which water power developments have been made and which are able to control the entire market, are, indeed rare. Steam competition may be expected for a great many years. This competition is present in its strongest form at the time a decision is being rendered as to the feasibility of a water power development. If the results of an investigation show that power can be produced by steam at even a somewhat higher rate than by water, decision will usually be rendered in favor of a steam plant. A water power plant, however, that can deliver power at a lower rate than a steam plant, will be able to obtain a large proportion of expansion and new industries, and such load as would require renewals of equipment until the capacity of the water power is reached. Fre-

quently agreements with existing power plants can be made which are advantageous to all concerned.

A form is given below which indicates the information which should be obtained in making a census of the power market. This census should cover all the area included in the economical range of the proposed development, which runs from a few miles to over 200 miles, depending upon the voltage used in transmission and capacity of plant.

POWER MARKET CENSUS

User	Place
Office	Date
Plant	

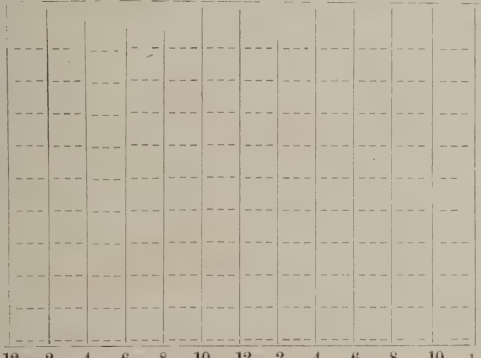
Local.

Source of Power	Surplus
Boiler (Producer) Capacity	Extension
Prime Movers	Condition of Plant
Generators	Management

Local or Central.

Connected Load	Central Station
Drive: Individual / Group / Shafting	Service
Motors: Type / Voltage	Estimated Unit Cost

Load Curve.	Remarks.



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The information obtained in this census will cover the prospective power requirements of existing plants. It does not cover the needs of new industries. Herein will lie the justification of the greater number of future water power developments. The demand for the cheap power necessary in economically producing materials or manufacturing articles of use to man will call for water power development. The vital question will not be whether new hydraulic power can replace existing power but whether water power will make possible new or more industries. The rapid development of the electrical art has made and is making great demands for power. No reference need be made to the ordinary uses of power, such as motor drive in industrial plants, lighting, and the like. Of the newer industries many offer most attractive fields for electric power. Some of these industries which are commercially successful are the manufacture of aluminum, calcium carbide, carborundum, caustic soda, chlorine, phosphorus, bisulphide of carbon, graphite, and nitrogen. To this must be added smelting, electro-deposition of metals and electro-refining.

Some of these industries may be taken to the source of power development, doing away with heavy transmission charges and losses. Others may be successfully carried on with power only a portion of the year. The power investigation should include a study of all probable fields with an estimate of the growth. It may be necessary to develop an industry before a development becomes feasible, or, what more frequently happens, the industry in search of cheap power brings about the development.

The results of the investigation of the power market may now be plotted in the form of curves, one showing the average load curve for a year; one, the same information for a day (Fig. 9) and another, showing expected growth. While these curves will usually be estimates, they should be sufficiently reliable to warrant proceeding with the investigation. Before actually starting construction, sufficient power should be under contract to, at least, verify the engineer's estimates.

A comparison now of the power chart modified by possible pondage or storage and the load curve will allow drawing definite conclusions as to the extent of the development.

(c) Where the load curve exceeds the power available or where a reasonable reserve is not assured, recourse must be made to auxiliary power. The use of auxiliary power allows working the water power development to a higher limit, since flood flows can

be used, falling back upon the auxiliary supply only at the times the power demand exceeds the power available. Here, again, the deficiency curves illustrated in Figs. 2 and 3 should be used to obtain the amount of auxiliary power necessary to bring the development to the contemplated capacity.

Having a measure of the auxiliary power required, its source should be investigated as follows:

(1) Development of adjacent water power projects with similar or different power characteristics;
(2) Installation of steam or gas auxiliaries;
(3) Combination with existing power companies;
(4) Arrangements with customers having power installations.

The natural tendency in the development of power is toward combination. Where two or more plants can combine in supplying one distribution system, a marked gain in reliability of service is obtained, as well as reduction in the number and amount of reserve apparatus. The system can be operated at higher efficiency in that different plants can be manipulated to best suit their power characteristics. In this study of auxiliary power an investigation should be made of other water power possibilities. Frequently adjacent drainage basins will show marked differences in power characteristics which, when combined in two or more plants feeding one system, provide a much improved characteristic.

The installation of steam or gas auxiliaries will follow best practise, being designed to meet the power deficiency, and situated so as to best serve the distribution system.

Frequently a water power development will be able to supply existing power companies at a rate below that at which they can produce it, but not sufficiently low to warrant duplicating the distribution service. In such a case agreements can be entered into, in which the existing power company will be furnished with power at a figure which will allow it to hold its plant as an auxiliary or reserve to the water power development.

Again, arrangements can be made with users of power, whereby they can afford to keep existing plants in repair and subject to call in case of failure of the water power. In this way, it may be possible to make up a considerable amount of the power deficiency due to occasional times of low stream flow, allowing a much greater average power output.

In the average development at its beginning, the power demands

will usually be such as not to exceed the power available and the need for auxiliary power will not be felt until the market has grown or been more fully developed. This should not, however, limit the investigation. The maximum capacity of a plant should be allowed for from the beginning as regards foundations and special construction for installation of the hydraulic machinery; the actual installation of the machinery waiting the increased demands for power.

V. PLANT DESIGN.

A tentative layout of the plant will have been obtained as a result of the investigation leading to a selection of site. This layout must now be made to conform to the apparatus actually needed to develop the economical capacity. It is not the province of the engineer actually to design particular machines, but to combine machines in such a manner as to give a satisfactory whole. The type and sizes of machines to fit local conditions must be prescribed. This is illustrated in the selection of turbines for developments with and without pondage. In the first case, where the economy of water is of importance, a turbine whose efficiency at all loads is high, is essential. In the second case, the efficiency at full load is of real importance, and not necessarily so at part loads when the water not used is wasted.

The combined capacity of the system should be such as to take care of the peak load to be supplied with a reasonable reserve. Individual units should be of such size as to allow following the load curve with machines operated at or near their points of maximum efficiency.

In the selection of turbines no definite limiting lines exist as to the fields of the different types. In low head developments the high speed single runner reaction type has advantages in that it holds up in power output over a considerable variation in head, and being of higher speed allows the use of a cheaper generator. With medium heads, efficiency at all loads becomes of more importance while the variation in head is not so great, making the high efficiency of the medium speed turbine desirable. Under high head, the low-speed reaction type or tangential water wheel may be used. Where the wheel is to be operated near full load the high full-load efficiency of the reaction type makes its selection desirable. Where the load varies, the comparatively flat efficiency of the tangential wheel makes its selection desirable. Under very

high heads, the tangential water wheel alone is suitable owing to structural features and the relatively low speed of this type.

In the selection of generators, the higher the speed, the cheaper the unit. The turbine selection is based to a certain extent on this requirement. The standard generating voltages are 2,200, 6,600 and 11,000. The choice being governed by speed and capacity of plant. Standard transmission voltages are 11,000; 33,000; 66,000; 110,000 and above, depending on the amount of power and distance transmitted.

The selection of auxiliary apparatus such as governors, switching and lightning protection apparatus, and transformers should be in keeping with existing practise, having care to obtain a certain system of standardization which will reduce costs in design, construction and, later, repair and operation.

Here, again, successful plants in operation will offer illustrations of what has been done and when due allowance is made for special conditions existing at the development under consideration, will assist in making a better development.

VI. COST.

The commercial success of any development depends upon whether it is financially feasible. A sufficient amount of power must be sold at a value which will net a reasonable return on the investment. This criterion should govern, and the engineer must not hesitate to report adversely on any development which fails to insure a satisfactory return. Throughout this discussion reference has been made to a certain procedure in each phase of the investigation, but emphasis must be laid on the element of cost in each and every item. Rarely is one solution of a detail of the development sufficient, others must be investigated and the relative advantages weighed against respective costs. In the development as a whole, usually more than one solution presents itself, and not until all reasonable lines of investigation have resulted in a financial estimate can a decision be rendered as to the feasibility of a development and as to its being the most economic development.

A large amount of cost data on water power developments is available, as well as numerous estimates as to cost per horsepower for initial investment and annual output. This information is of such a variable nature, due to the different characteristics of the many developments, that its use is not warranted.

Having tentatively fixed upon the different features of the de-

velopment, a detailed estimate of the cost should be made basing it on local conditions. The usual items covered in this estimate are:

1. Real estate;
2. Water rights;
3. Dam, including excavation;
4. Headworks, including canals or penstocks;
5. Tail race;
6. Power house;
7. Hydraulic equipment;
8. Generators, exciters, and accessories;
9. Switchboards and apparatus;
10. Transformers;
11. Protective apparatus;
12. Transmission line, including right of way;
13. Sub-stations;
14. Distribution system;
15. Office, storage, and service buildings;
16. Miscellaneous, as roads, raising bridges, etc.;
17. Engineering and contingencies;

When an auxiliary power installation is included, the additional items appear:

18. Real estate;
19. Power house;
20. Boilers and accessories;
21. Turbo-generators and accessories;
22. Switchboards and apparatus;
23. Miscellaneous, as water supply, railroad spur, etc.;
24. Engineering and contingencies.

A summation of these detail estimates will give the total estimated capital cost. This indicates the amount of money which must be raised to finance the undertaking and may be so great as to prohibit or delay the development. This amount does not, however, allow a comparison with the existing cost of power in the locality. It is now possible to compute the annual cost of the water power, it being the sum of certain fixed charges and operating expenses. The fixed charges are:

1. Interest on capital cost;
2. Taxes;
3. Insurance;
4. Depreciation;
5. Miscellaneous, such as do not vary with output.

The operating expenses are:

1. Labor; 2. Supplies.

Of these items the interest on the capital will be large, owing to the large investment involved. Other expenses will be lower than in a steam plant, with the possible exception of labor due to the patrolling and maintenance of long transmission lines. Having the annual cost estimate, it is possible to compute the cost per

horsepower hour output. From the annual load curve determine the ratio of average power to maximum output, known as the load factor. The cost per horsepower hour will then equal

$$\frac{\text{Annual cost}}{(\text{Maximum output})\ (\text{Load factor})\ (8760)}$$

In determining the cost of existing power in the locality, a corresponding estimate must be prepared in the absence of authentic information. The cost per horsepower hour of steam power admits of close approximation, owing to the abundance of valuable data on costs of steam power. If the water power project calls for steam auxiliaries, an estimate in keeping with the locality will already be prepared, which, when altered to suit existing plants, will give the information desired.

Having unit costs of the proposed development and existing plants, conclusions as to its financial feasibility may be drawn. When the water power cost exceeds the steam cost, present conditions would indicate the construction of a steam plant to be advisable. When the water power cost is only slightly under that of the steam power, the disadvantages of water power are usually sufficient to render the development inadvisable. An especially favorable market may make the development advisable.

Where costs are unfavorable to the water power development other solutions of the problem should be investigated, such as the development of different amounts of flow, use of more or less auxiliary power, development of less head, and the like.

VII. REPORT.

A report should be prepared giving a clear, concise and definite statement of facts and recommendations, with the necessary hydrographs, drawings and data to show conclusively the facts as set forth.

The outline of this report is given:

I. Water—
- (1) Drainage area.
  - (a) Extent.
  - (b) Physical characteristics.
- (2) Rainfall and run-off.
- (3) Stream flow.
  - (a) Duration and deficiency curves.

II. Location of plant—
- (1) Selection of site.
- (2) Description.
  - (a) Survey.
- (3) Layout of plant.
  - (a) Foundations.
  - (b) Methods of construction and supply.

III. Available fall—
  (1) Effective head chart.

IV. Power—
  (1) Available from stream-flow and head.
    (a) Pondage and storage.
    (b) Power curves.
  (2) Market.
    (a) Existing power.
    (b) Future requirements.
      (I) Growth of existing market.
      (II) New industries.
  (3) Load curves.
  (4) Capacity or extent of development.
    (a) Auxiliary power.

V. Plant Design—
  (1) Selection of equipment.
  (2) Design of power house, dam, accessories.
    (a) Drawings.

VI. Cost—
  (1) Capital cost.
  (2) Unit costs.
  (3) Comparison of unit costs.

VII. Conclusions and recommendation.

In the investigation of a water power development one is prompted with the desire to conserve the natural resource involved, stopping the evident waste of energy. A failure to use this energy, however, will not affect the regimen of the stream or decrease the amount of energy. It will remain awaiting the time when economic conditions will warrant the development. Only by a careful investigation can the necessary information be obtained which will allow rendering a decision as to whether economically a development is possible. Where private interests take up an investigation, it is reasonable to expect that conditions are favorable to a development, otherwise the expenses of investigation will not be warranted. In such a case a desire for profit, sometimes in excess of that proportional to the risk involved, holds up a development which should be made.

Governmental investigation, if thorough and accurate, should do much to further water power development. Where a favorable report is rendered, it should attract the necessary legitimate capital necessary to proceed with a sound development. On the other hand, an unfavorable report should prevent an unprofitable venture.

The data obtained and conclusions drawn, even when unfavorable, are not wasted, but should form the basis of additional investigations to be made at later dates when economic conditions have changed. Likewise, an investigation of this kind will show where sufficient data is not obtainable, as for rainfall and stream flow, and should lead to the establishment of stations or other means of obtaining the necessary information.

In conclusion, a development capable of rendering efficient and economical service as well as a proper return on the investment is possible only after a thorough and proper investigation.

Floating Concrete Caissons. (Foreign and Domestic Development.)*

BY

Mr. HARRISON S. TAFT, *assisted by* Mr. O. D. JONES

INTRODUCTION.

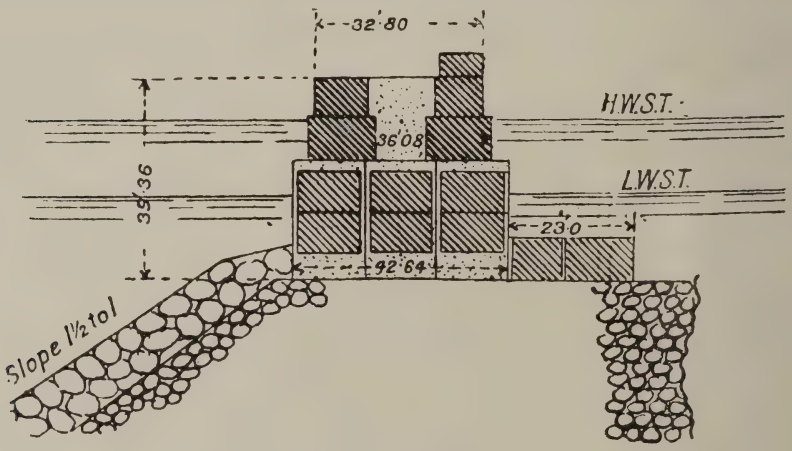
Apparently in overlooking the opportunity and advantages to be derived in using floating concrete caissons in port and other subaqueous work, the harbor development engineers of this country have perhaps been neglecting a type of structure that has been widely and successfully used by foreign port engineers since 1905. It is possible that the small amount of attention which has been given to an extensive development of our shipping ports during past years has not called for any special treatment of the subject. It is also possible that past failures in the use of cement under salt water conditions have prejudiced conservative concrete engineers against the use of such a material when in contact with sea water. Then, again, certain legal considerations may have operated against the development of floating concrete caissons in hydraulic engineering in this country.

Whatever have been the real reasons for the antipathy of the American engineer toward taking up the development of floating concrete caissons, in view of the vast sums that are now being spent in making our harbors and shipping facilities equal to the most modern of foreign ports, it would seem that such a type of structure might be made to play an important part in such work. Not only can they be built in the dry and thus become thoroughly cured before coming into contact with the sea water (a most essential feature in the use of cement subjected to salt water action), but their economy over old time methods of such uses of concrete is marked.

In order to learn of the real value and worth of such a type of structure in future development of our ports it is perhaps fitting to review what other nations have been doing in the use of floating

*Contributed by Mr. Harrison S. Taft, Contracting Engineer, Central Building, Seattle, Washington.

concrete structures, especial emphasis being placed upon floating concrete caissons in this discussion. While such a review may not determine the origin of the art, it does seem apparent that the actual practise of using floating concrete caissons has been carried out simultaneously in Europe and America. Though United States patent papers covering "Reinforced Concrete Caissons for Use in Hydraulics" were granted to Col. W. V. Judson, Corps of Engineers, U. S. Army, in 1901 and 1909, the records of foreign technical papers, etc., indicate that a number of years before Colonel Judson made a practical application of his idea, floating iron caissons, constructed partly of concrete, were used by foreign engineers in the construction of their breakwaters, and other



West Jetty, Bilbao.

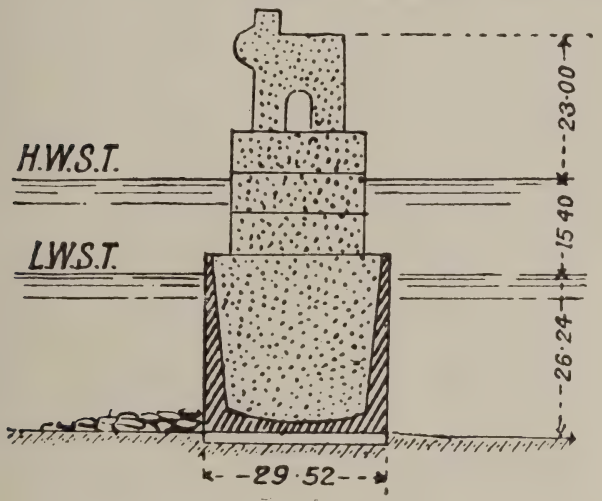
subaqueous structures, thus perhaps opening up the question as to who was the real originator of such a mode of construction, the use of reinforced concrete having been but a detail improvement over and above the type hitherto used.

FLOATING IRON CAISSONS.

Since the idea of using floating concrete caissons seems to have been a step in advance from the use of iron caissons, it is fitting to make note of a few undertakings with such a type of structure, especially those structures which it has been plainly stated led to the adoption of full concrete structures in the subsequent undertakings. For, as will be soon noted, the successful use of floating iron caissons in the construction of the west jetty at Port of

Bilbao, Spain, during 1895-1902 led to the adoption of full concrete caissons in the construction of a breakwater and sea wall at Barcelona, Spain, in 1905.

The first of the iron caissons used at Bilbao was constructed in 1895, the successful results obtained with this pioneer structure resulting in the same system being used throughout the entire work. Each of these caissons was approximately 42½ feet long, 23 feet wide and 23 feet high. After being built on the shores of the port they were put afloat and ballasted with concrete to a depth of 5 feet from the bottom. They were then towed to and sunk into position upon a rubble foundation by means of water ballast. Twelve premoulded concrete blocks, each about 13 by 9 by 8 feet,



Jetty, Zeebrugge.

containing 40 cubic yards concrete (equal to about 80 tons) were immediately placed in each caisson in order to obtain sufficient stability against wave action in the shortest space of time; a much quicker operation than if concrete placed en masse in situ had been used. The voids between the sides of the iron caissons and the blocks were then filled with grout. On top of the substructure thus prepared was built a superstructure in way of a sea wall. The time of placing and filling a caisson was about thirty-one hours. This undertaking appears to have been the pioneer in the use of such a mode of construction.

At a still later date (previous to 1900) the same mode of construction was used at Zeebrugge in the North Sea, with the ex-

ception that the iron caissons rested directly upon the bottom of the sea, and not on a rubble fill. The iron caissons in this undertaking were 82 feet long, 29½ feet wide, 28½ feet high, and weighed about 4,500 tons when filled with concrete. The caissons consisted of an iron frame built of angles and tie members, the whole covered with a steel plate so as to form a concrete form, as it were, of the proper shape, the concrete being poured into the form thus built, the metal form becoming a part of the permanent structure. The thickness of the sides of the complete structure was 3 feet at the base, 1 foot at the top. The caissons were floated to, sunk into position and then filled with concrete placed en masse in situ.

Though the authors found references to "The launching of concrete and masonry blocks in repairing jetties in 1901," they were unable to obtain detail information concerning the work. From the title it would appear that some type of a floating concrete caisson might have been used. Still, from the few cases cited, it is self-evident that floating concrete caissons were, as already stated, simply a step in advance of floating iron concrete caissons, a step that would naturally follow with a wider development of uses of concrete.

Though at first the difficulties of launching a concrete structure void of any reinforcement would seem insurmountable, it appears (as will be noted further on) that the uncertainty as respects the floating capacity of a cellular concrete structure of large size was the prime factor that delayed the use of floating concrete caissons in hydraulic engineering, the launching problem having been solved by the conception of a lifting-depositing dock on the part of English engineers.

SPAIN.

Barcelona. While the conception of using floating reinforced concrete caissons in hydraulic engineering may have originated in America, the first practical use of such a mode of construction seems to have been made in a foreign country, namely, Spain.

As stated by Don Julio Valdes y Hamaran at the International Congress of Navigation held at Milan in 1905, the design adopted for the Barcelona (Spain) breakwater was "derived from the type so successfully used at Port of Bilbao" (Spain). A riprap substructure was first constructed and 80 tons concrete blocks thrown in at random, "raising the substructure on the outside

from about 30 feet below to about 6½ feet above sea level.'' On the inside was built a wall ''composed of monolithic concrete caissons, *forming the last word in this sort of work.*'' The space between the concrete caissons and the pel-mel concrete blocks was filled with riprap of large stones, a double tier of concrete guard blocks being set on top of the fill. The quay and shelter wall of the breakwater rest on the monolith caissons. The plans for this structure were approved of in 1903 and the work begun soon afterwards, being in full swing during 1905. From the above it appears that the first use of floating concrete caissons was on the part of Spanish engineers.

As noted in the *Proceedings of the International Congress of Navigation,* held at St. Petersburgh in 1908, the floating concrete caissons used in Barcelona in 1905 were ''not made of armored concrete.'' They were made ''without any kind of strength at their base, in the long walls or in the intermediate bulkheads, thus avoiding decomposition of the armored concrete and obtaining a maximum economy in first cost by not resorting to expensive material.'' The caissons were about 46 feet long, 20 feet wide, 23 feet high. The cement used was ''Grippe Cement from Teil.'' The bottoms of the caissons were made of practically a 1-2-4 mixture, the walls of a 1-1½-3 mixture. The walls were also given a coating of 1-1½ mortar in order to increase their water-tightness.

From information available it seems that the caissons used at Barcelona were built on a sort of building stage and then put afloat by means of the lifting-depositing dock, spoken of above, and as will be further described in discussing the work at Kobe, Japan. The caissons were given one month of curing before being put afloat and then moored in the harbor for various periods in smooth water awaiting a favorable opportunity to set, though no doubt some were set immediately after launching. The time required to manufacture one caisson has been stated to have been eighty-four hours; the time of towing to their final resting place and the sinking thereupon about three hours. The method used in sinking was by water ballast admitted through valves in the sides of the caissons. It appears that in some cases large sized concrete blocks were used to fill the settled caissons with as in case of the Bilbao work already spoken of; at other times large sized stones were used, the water pumped out and a grout fill pumped in to fill the voids. Then, again, concrete placed en masse in situ is said to have been employed.

As is usual in pioneer work, it is perfectly plausible to imagine a variety of schemes having been used in the work, each an improvement upon the other all leading to a successful conclusion of the entire undertaking, detail information upon which is perhaps not readily obtained by the American engineers.

Whereas the caissons employed in the pioneer usage were of a somewhat small size compared to similar structures used now-a-days, this fact was foreseen, for, as Don Hamaran stated in 1905, "... it is believed that the system would be perfectly applicable with larger dimensions. Furthermore, the net cost per unit would be less as the use of cement and the cost of transportation of the blocks, which are inversely proportional to the volume of the latter, would be reduced."

HOLLAND.

Rotterdam. The most extensive use of reinforced concrete caissons in harbor development work appears to have been made by the Hollanders, who perhaps are, from necessity, the most famous hydraulic engineers in the world. The first use made by them of such a type of structure was in 1905, in the rebuilding of a river wall at Rotterdam that had become badly damaged. In fact, the original wall had for some unknown reasons split in two and been shoved out of line for a distance of some 675 feet; even to the extent of some 5 feet at one point in the wall.

In making the repairs rows of piles were first driven along the face of the old wall and capped with heavy timbers below the water line. Six reinforced concrete caissons were constructed in a dry dock, given three weeks of curing and then put afloat like a big barge. After being kept afloat for about two months, the caissons were towed a distance of some 3 miles to the location of the damaged wall. After being properly located they were sunk to position upon the prepared piling foundation in such a way that two-thirds of the caisson rested upon the base of the old wall; one-third upon the wood piling. The caissons were then built up to the height of the old wall.

The first one of these caissons was about 75 feet long, 18 feet wide and 4½ feet deep with partitions about 6 feet apart; thus making a cellular structure of it. The success obtained with this first attempt was marked and justified the engineers in using the same method of procedure in repairing the rest of the wall, the other five caissons being 120 feet long and 30 feet wide. These

caissons were of a 1-2-2 mixture with the outside surface coated with tar.

The unqualified success attending the use of floating concrete caissons in making these repairs led the engineers of Rotterdam to build full depth caissons which would rest directly upon the sandy soil without any subpiling, a type of structure that has become the standard practise for their river walls.

In undertaking the construction of six full depth caissons in building some 615 feet of wall in 1906, the engineers first experimented with two such structures, building them simultaneously in a dry dock. The caissons were 60 and 75 feet long, respectively, each being 30 feet high. In order to obtain proper stability, it was necessary to place ballast in the two caissons before launching. Since the floating draft was more than the depth of water over the dock's sill, it was necessary to use pontons in floating the concrete caissons.

After being put afloat the caissons were towed to their final resting place in the wall and sunk onto the prepared sandy bottom. The experience with these two caissons led the engineers to consider the feasibility of building the caissons only to such a depth as to guarantee floating stability when first put afloat, their upbuilding to take place afterwards. These two caissons also led to a more satisfactory design for the following caissons and proved conclusively that large concrete structures of a cellular construction could be kept afloat for a long time.

The four remaining caissons in this second wall were designed for a length of 120 feet, a depth of 30 feet and a width of 21 feet, the walls being built with a batter. The main body of these caissons rested upon a heavy concrete slab, built monolithic with it, 29 feet wide, thus forming a shelf, as it were, some 4 feet wide on each side, beyond the sides of the main body of the structure. The ends of each caisson were so built as to form a dove-tail joint with its neighbor. The caissons were divided into twenty compartments by means of nine athwart partitions and one longitudinal one.

In constructing these four caissons a pit was dug in the river bank, covered with 9 feet of sand and a platform made by driving wooden piling into the fill. The caissons were first built to the height of some 17½ feet and then put afloat, having a draft of about 11 feet with a freeboard of about 6 feet. They were then built up to a total height of 30 feet, corresponding to a

draft of 22.5 feet, ballast being inserted to provide the necessary stability. The caissons were then towed to their proper place in the wall and sunk into position upon the prepared bottoms by means of water ballast, the entire work being carried out with great success.

After the sinking operations had been completed, the cells of the caisson were filled with concrete and sand, concrete being used for the front row of cells, sand for the back row. If the front row of compartments had been filled with sand there was danger that it might in some way work out of said compartments and thus destroy the stability of the whole wall against sliding. As the concrete fill was placed in the dry the outer walls of the caisson, as well as every other cross wall, had to be designed for a pressure head of 33 feet.

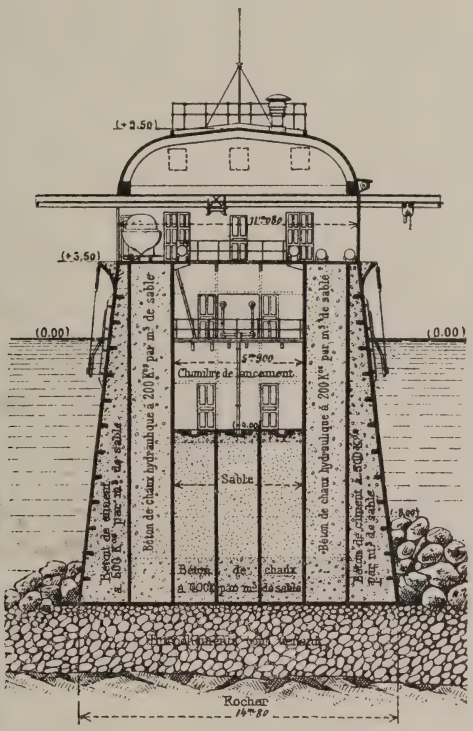
Since the caisson had but 1½ feet of freeboard above low water when sunk into position and the water rose to 3½ feet above them at high water, it became necessary to place an iron deck over four adjoining compartments—made water-tight by hemp packing—the decking being provided with four hatchways, one to each compartment. The four compartments were then pumped out in such an order and at such a rate as not to create too great a difference in water level and thereby avoided any excess pressure upon the interior partitions. The two front compartments were then filled with concrete at the same time—the two rear compartments being filled with water to preserve an equal pressure on each side of the longitudinal bulkhead. After the concrete had been completed the two rear compartments were filled with sand dumped into the water in the compartment. The main wharf wall was then erected on top of the completed caisson.

The third attempt with floating concrete caissons at Rotterdam followed soon after the completion of the above six full-depth caissons, the same mode of construction being used as in the preceding case. The wall in this third undertaking was 4,800 feet long. Since it was possible to have under actual process of construction four full-depth caissons at the same time as four other caissons were in the process of setting in the wall, each caisson being 120 feet long, 960 feet of the wall was in the making at one time, a rate not possible with the ordinary modes of building walls.

A still later use of concrete caissons at Rotterdam was shown in 1911, in a further extension of their river walls. In this case the

type and mode of construction used in the building of the 4,800 foot wall was adopted, the caissons being 134 feet 6 inches long, 32 feet 9 inches wide with a base of 31 feet 10 inches.

Since the Rotterdam engineers have standardized the type of their caissons it is reasonable to suppose that all future river wall work done at Rotterdam will be similar to the above described methods and that said method can be adopted with a guarantee of success by other engineers.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Béton de chaux à 300 par m³ de sable
Rocher
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Transverse Section through Station for Testing Torpedoes, near Toulon, France.*

DENMARK.

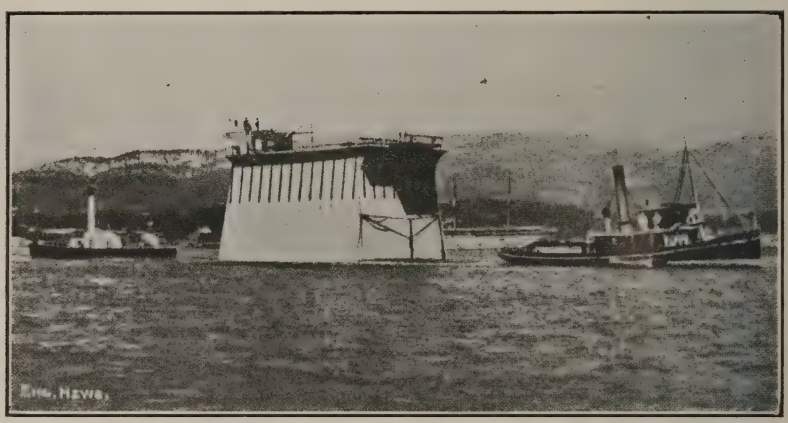
Norre-Sundby. At this harbor in Denmark resort had to be made to floating concrete caissons in the construction of a dock 725 feet long, on account of the poor soil and the teredo. The methods used were similar to the methods used at Rotterdam, with the exception of the size of the caissons. The Denmark caissons were 32

*Reproduced, by permission, from *Engineering News*, July 8, 1909.

feet 9½ inches long, 25 feet high, and contained five compartments, there being four athwart partitions and no longitudinal ones. The fill material was of sand. A concrete wall was built along the upper half of the caisson above low water level.

FRANCE.

Toulon. The first attempt with floating concrete caissons on the part of the French engineers seems to have been in connection with the construction of a torpedo testing station near Toulon, France, in 1909. The caisson was of a truncated pyramid shape, was built in a drydock, put afloat, towed to and sunk in its final position. The base of the truncated structure itself was 77 feet long by 48½ feet wide. The platform slab upon which the main structure



Torpedo Station, near Toulon, France. Towing the caisson to final site.*

rests was 77 feet long but 55 feet wide, thus forming a shelf outside the truncated structure of some 3¼ feet along each side. The height of the truncated structure is 51 feet, 11½ feet of which is above the water level, the remaining 39½ feet being submerged. The sides and ends have a tumble-home with a flared-out shelf at their top so as to cause the waves to return on themselves. Thus the top of the structure is but 70 feet long and 36½ feet wide.

The caisson is divided into seventy-seven compartments by means of ten athwart and six longitudinal concrete partition walls, the compartments being filled with a lean concrete and sand ballast to weigh the caisson down. The bottom slab, the outside walls and the interior partitions are but 6 inches thick, properly rein-

* Reproduced by permission from *Engineering News* of July 8, 1909.

forced. The base slab is built with a camber of 2 inches transversely. On the top of the caisson is located a superstructure of a design suitable for the purposes of its special uses.

In building the caisson a dry dock was made use of. First the 6-inch slab was poured, then the exterior and interior walls were brought up, the former to their full heights, the latter for a height of but 10 feet. After the caisson had been put afloat, the interior walls were brought up to a height of 37½ feet. A concrete fill was also put in the lower part of each compartment to a depth of 6½ feet, giving a draft of 26½ feet corresponding to a displacement of 2,800 tons. After being properly protected and internally braced, the structure was towed some 22 miles to its final resting place at a speed of about 1½ miles per hour without any difficulty. The caisson was sunk by water ballast to its final resting place on a prepared bottom. The different compartments were then one by one pumped out and filled with a suitable concrete fill. The fill in the compartments next to the shell was of a lean cement concrete carried their entire depth. The next set of compartments had a lean lime concrete fill, also carried their full depth. The interior compartments were filled with sand to the level of the working platform and then covered with a cement floor slab. A rock fill is provided around the outside of the caisson at its base to prevent scouring by wave action.

Paris. In building a pier for a railroad bridge over the Seine in 1911, four reinforced concrete caissons were first built on the river bank, launched at the end of six weeks of hardening, towed to their final location and then sunk to position. Each caisson was 34½ feet long, 14½ feet wide and 14 feet high, well reinforced in each direction. The upstream ends were rounded. The walls were 6 inches thick at the bottom, tapering to 5 inches thick at the top. The caissons were built with a temporary wooden base which could be knocked out when the caissons were in their final position for sinking. After being sunk the interior of each caisson was filled with concrete, the fill resting upon 12-inch diameter concrete piles.

Havre. In constructing the West Dike at Havre, floating concrete caissons were built on land, set afloat, towed to and sunk into position on top of a rock embankment built above the bottom of the harbor. The length of the dike was 4,740 feet. Each caisson was 82 feet long, 19.68 feet wide and 18.86 feet high. The lower third of the caissons had vertical walls, the upper two-

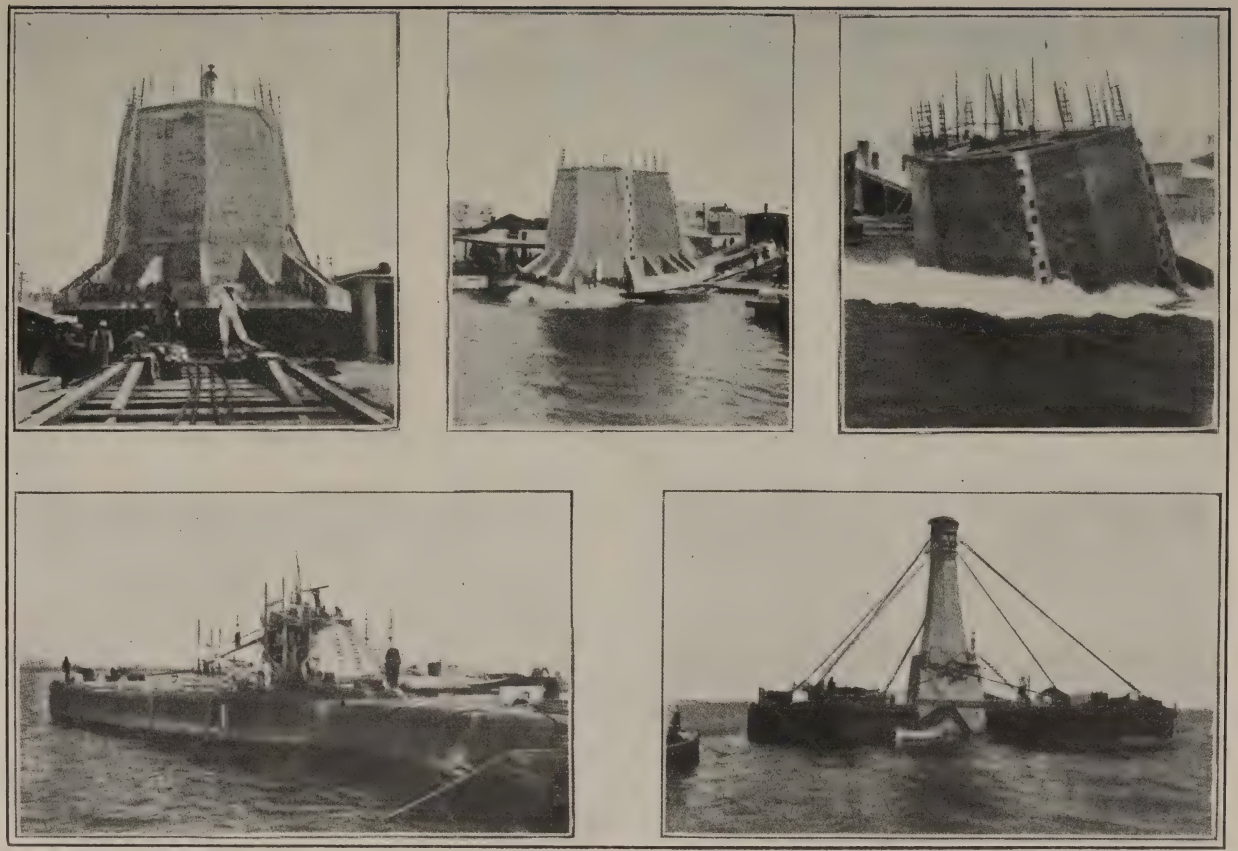
thirds being built with a tumble-home so that the width of the caisson on top was about 11 feet. A longitudinal bulkhead, located 6 feet from the face wall and having the same general shape as the outside walls, divided the caisson into two compartments. After being sunk into position the outer compartment was filled with concrete, the inner compartment with earth. On top of the caissons a regular sea wall was constructed of brick work.

EGYPT.

Alexandria. During the first half of 1911, a concrete lighthouse was constructed on the shore of Alexandria Harbor, Egypt, in the form of a reinforced concrete caisson and floated out to its final position some 4 miles from its place of construction. The base of the structure consisted of a seven-sided slab, 8 inches thick, reinforced with one 12 by 24 inch longitudinal beam and five 8 by 16 inch transverse beams. The axial dimensions of the base were 37 feet by 41 feet. On top of the slab a hollow tower some 45 feet high, having tumbled-home sides and of the same general shape as the base, was erected. The shell or sides of the base tower were 6 inches thick, being strengthened by 8 by 12 inch vertical webs and 6 by 6 inch horizontal ribs. At the center of the tower was located a 12 by 12 inch column with 8 by 8 inch struts extending out to the sides. Since the base of the tower was set in from the edge of the horizontal base, making a shelf, as it were, 5 feet wide, it was necessary to provide a series of brackets on the outside of the tower itself at its intersection with the base; also to construct a rim or beam all around the outer edge of the horizontal base. A lantern tower, 36 feet high, was constructed on top of the base tower. This lantern tower was 12.3 feet in diameter at its base and 8.2 at its top, being well braced by 6 by 6 inch and 8 by 8 inch bracing struts and beams. The thickness of its sides was 4 inches.

The whole structure weighed about 1,700 tons with a displacement of some 600 tons, resulting in a load of 1,100 tons on the sands. A 1-2-4 concrete with no waterproofing material was used.

In building this structure, the flat base and 24 feet of the caisson tower were first constructed on land and then launched. The caisson was then built up to its full height while the structure stood alongside of a wharf. The lantern structure was also cast on shore, placed on top of the lower structure and made secure thereto.



Lighthouse Caisson and Tower, Alexandria, Egypt, showing (1) Caisson on Slip; (2 and 3) Launching, first and second stages; (4) Completion of superstructure; (5) Steadying of floating caisson between two lighters, front view. Reproduced from *Annales des Ponts et Chaussees*, 1912.

The whole structure was lashed between two sea-going barges with the tops of the upper and lower towers well guyed to the outer gunwale of the barges. An attempt was then made to tow the entire structure to its place of final location. When about half way on its journey, the upper tower broke loose and fell over onto one of the barges without injuring the tower to any great extent. More damage is said to have been done to the caisson by the fall than to the tower, the latter being set upon shore as a land beacon. When the caisson finally reached its place of depositing, it was sunk onto the prepared bottom by filling with water. The inside was then filled with concrete placed in the wet to such a height as to preserve the stability of the structure when pumped out. After the caisson had been unwatered, the rest of the interior concrete was placed. The lantern tower was then built in situ and the structure as a whole completed. Though the tower was in heavy collision with a 4,000-ton steamer it was not damaged nor was its light put out at the time of the collision, thus going to show what an amount of lateral pounding concrete will stand when properly made.

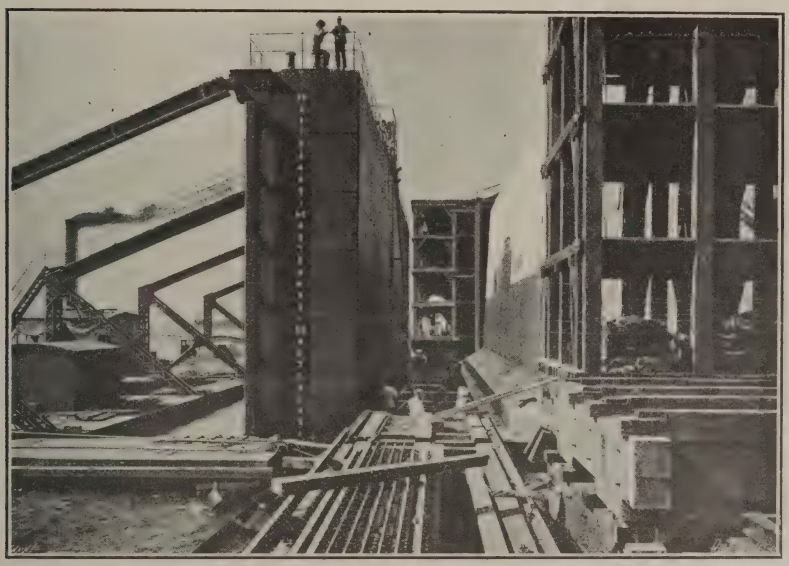
ITALY.

Genoa. In the construction of a set of piers at Genoa, Italy, it was necessary to make use of floating concrete caissons in the construction of two of said piers. Each caisson was 17 feet 3 inches by 13 feet 11 inches and 36 feet high. The lower part was built with a cutting edge and a working chamber 6 feet high. The concrete walls were strengthened by two vertical webs on each side of the caisson with a series of struts across the interior from web to web. After being put afloat and towed to their final resting place in the harbor they were sunk into the mud by excavation in the air.

RUSSIA.

Whereas the Russian engineers seem to have made considerable study of the application of floating caissons to harbor development work, the authors have but one record of the actual use of such a type of structure on the part of said engineers. Still, it is of interest to note in passing that in Russia as well as at Valparaiso and Stolpmünde, bottomless concrete caissons "set by means of metallic pontoons applied to the sides of the caissons" have been used. But such types of structures are foreign to the subject of this paper and so will not be dwelt upon.

Tuapse. The only record that the authors have of the actual use of floating concrete caissons on the part of the Russians is in connection with the construction of a jetty at Tuapse on the Black Sea. The length of the caissons was 56 feet, the width 21 feet, with heights of 6, 12, 9 and 22 feet, respectively. The weight of the largest caisson when empty was about 268 tons; when filled, about 1,600 tons. They were divided longitudinally by two partitions, the outer compartments being filled with a lean concrete, the inner compartments with the same material less the cement.



Depositing Dock with its Pontons inserted beneath Caisson. (Kobe Harbor Improvement; reproduced from *The Engineer,* London, of March 24, 1911.)

The caissons rest upon a riprap base and so form the vertical part of the breakwater or jetty.

JAPAN.

Kobe. The most extensive use that has apparently been made of floating concrete caissons outside of Rotterdam is at Kobe, Japan, where large harbor improvements are under way at the present date. In the construction of four new concrete piers, reinforced concrete caissons—some 119 feet long, 35 to 40 feet wide, 24 to 30 feet high, divided into twenty open-top cells by interior partitions and weighing from 1,900 to 2,400 tons each—are being built

in the dry on a pile structure on the shores of the harbor. After sufficient curing the caissons are lifted off their pile foundation by a specially designed depositing floating dock, carried into deep water and put afloat by sinking the depositing dock and withdrawing it from underneath the concrete caisson. The caissons are then towed to and sunk in their proper places in the quay wall. When afloat, the caissons drew about 25 feet. The exterior walls of the caissons are on the average a little over 1 foot thick, being built with a taper. The interior walls are of two different thicknesses, alternately 8 inches and 11 inches.

The land foundation upon which the caissons are built consists

The Dock and Caisson withdrawn from the Building Stage (Kobe Harbor Improvement; reproduced from *The Engineer,* London, of March 24, 1911).

of seven groups of piling 15 feet apart, there being three rows of twelve piles in each group. The piling is properly capped with heavy timber running athwart the caisson, so as to permit the fingers of the lifting dock to be inserted under the caissons between each group of piling. The cost of the lifting dock was about $27,600.00. It takes two hours to put the caisson afloat.

Three building stages are used in the Kobe work, with an output of twelve caissons a year; three months being required to mold, mature, and put afloat each caisson. For building the seventy-nine caissons seven years will be needed. If the caissons had been

built on a floating dry dock it is estimated that twenty years would have been needed to finish the work or the use of five such docks for a period of seven years. Thus, while the lifting dock is used but at intervals it is a far cheaper mode of construction than any other possible method.

The original conception of a "lifting-depositing dock" is due to John Standfield of England, who has taken out English patents upon this type of structure. Whereas the idea at first did not find much favor, the principal reasons and the only one, as it appears, against such a mode of operation, was the doubt as to the possibility of building a concrete caisson that would have floating

The Dock carrying a caisson on its Pontons (Kobe Harbor Improvement; reproduced from *The Engineer*, London, of March 24, 1911).

capacity. As above stated, the first application made of the idea of a "lifting-depositing dock" was at Barcelona, where the port engineer used the device "for picking off the depositing stages the ferro-concrete blocks which formed the extent of the harbor of Barcelona." From the success obtained at Kobe it would appear that the system of construction used by them is one worthy of deep consideration on the part of all harbor engineers.

CANADA.

The introduction of floating concrete caissons in Canada seems

to have been the results of the work of Mr. Tromanhauser, who, in 1910, developed a type of caissons having a wooden bottom with concrete side and interior walls. While the use of wood may be permissible in fresh and non-teredo infested waters, it would seem that such a use of timber would not work out favorably to such undertakings in teredo infested waters unless the wooden bases were so well sunk into the soil as to preserve them against attack from the worm.

The first work done in Canada with floating concrete caissons by Mr. Tromanhauser was in the building of the foundation wall of a large building at Goderich, Ontario, in 1910. Four wooden

The Caisson Afloat, being Towed by a Tug (Kobe Harbor Improvement; reproduced from *The Engineer*, London, of March 24, 1911).

caissons 54 feet long, 14 feet wide and 5 feet deep were first constructed and launched. After launching, the concrete side and cross interior walls were started from the floor of the wooden caisson and built up 2 feet at a time to a height of some 20 feet. The interior partitions divided each caisson into six compartments 9 feet by 14 feet each. To insure good work, the fresh concrete was always kept above the water level until hardened. In order to maintain stability, it was necessary to place rock ballast in the compartments of the caissons as the walls were carried up. After the proper height had been reached, the semi-concrete floating

structures were towed to their position in the sea wall and sunk by admitting water through valves in the lower part of the structures. After all the caissons had been sunk to place, they were filled with rock ballast and the decking put in place.

Following up his first successful attempt with floating concrete caissons Mr. Tromanhauser made use of the same principle for the foundation of a grain elevator tower at Goderich in 1912. The caisson was 30 feet square by 24 feet deep and contained nine compartments. The outer walls of the caisson were 12 inches thick, the partition walls 10 inches. The same mode of construct-

Water-tight Ponton Bottom (Patented, Tromanhauser), as used at Goderich, Ontario.

ing the crib as in the case cited above was used, and the whole structure sunk onto a prepared rock bottom by filling the interior compartments with sand and gravel. Piles were also inserted in the fill so as to transport directly the weight of the tower to the rock bottom. A concrete slab was placed over the top of the filled caisson and the concrete tower erected thereupon.

*Goderich Breakwater.** As the direct result of Colonel Judson's success with floating concrete caissons on Lake Michigan, the Canadian Government decided to adopt his system of construction

*For cross section of this breakwater, see Fig. 5, opposite page 180.

in the building of a long breakwater at Goderich, on Lake Huron, Ontario. This work was started in the fall of 1911 with the building of three concrete floating caissons, each being 100 feet long, 26 feet high, with a width at their base of 35 feet but only 30 feet at their tops, thus giving a tumble-home of 2½ feet on each face. The caissons were divided into twenty-four compartments by means of two longitudinal and seven cross concrete partitions. These three caissons were sunk in the inner harbor at Goderich during the winter of 1911-1912, being refloated in the spring of 1912 and put in their proper place in the breakwater along with three other caissons that were constructed during the spring of

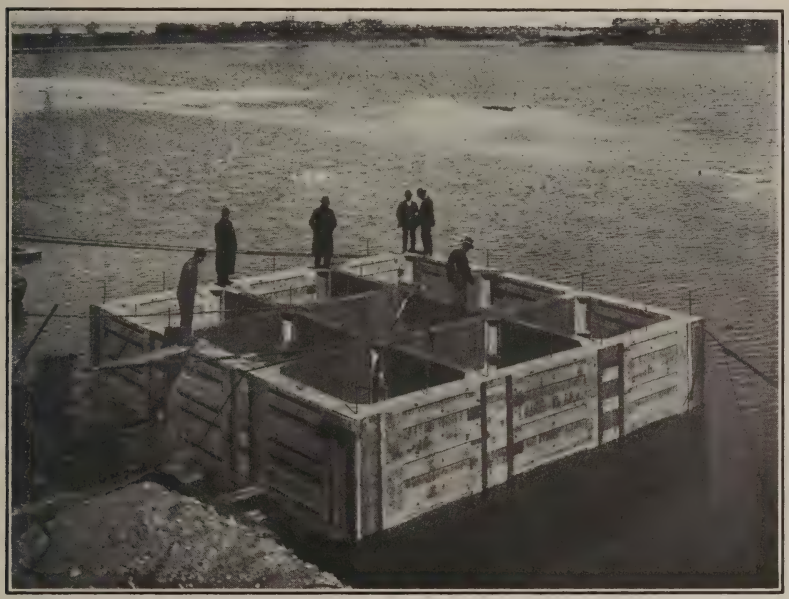
Completed Ponton (Patented, Tromanhauser) as used at Goderich, Canada.

1912, thus making 600 linear feet of work completed during one summer.

In the construction of the caissons a form of floating scow-dry-dock was adopted by the contractors. This dry-dock consisted of a specially constructed scow, 114 feet long, 44 feet wide, with sides 11 feet high, the bottom of the scow being built with special care as regards its use. One end of the scow was so built as to swing outwards and thus permit the concrete caisson to be floated out when built to a proper depth.

The base of the concrete caissons, properly reinforced, was first poured directly upon the deck of the scow. The interior and exterior walls were next brought up 3 feet at a time simultaneously.

care being taken to secure a water-tight joint between each pouring by means of grout. By the time the concrete walls of the caisson had reached a height of 14 feet, the top of the sides of the scow-drydock were almost submerged. By filling the space between the sides of the concrete caisson and the walls of the scow-drydock with water and by placing heavy weights therein, the scow-drydock was parted from the base of the caisson and sunk onto the bottom of the harbor. The end gate was then opened and the caisson towed out, floating on its own bottom. The further upbuilding of its walls was then proceeded with as in case of an ordinary



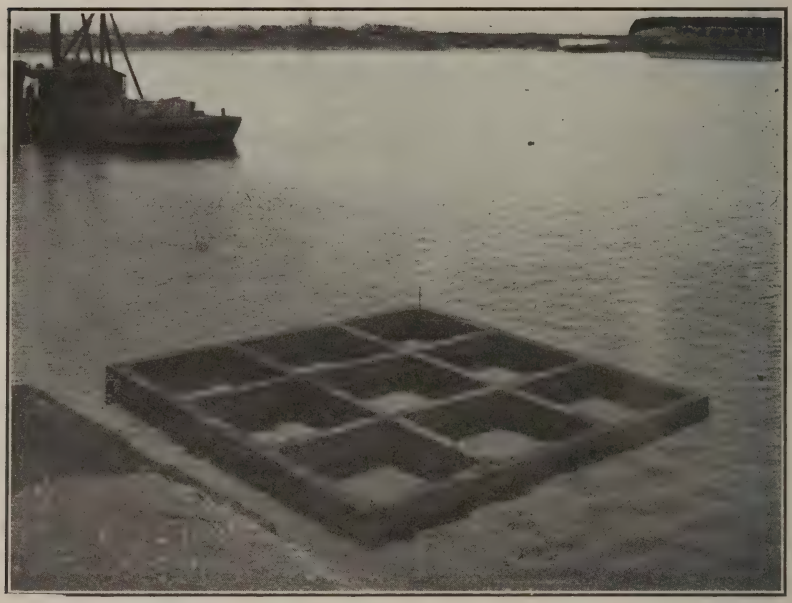
Completed Crib being sunk on tower site (Western Canada Flour Mills Co., Goderich, Canada).

wooden caisson. The weights being removed from the sunken scow-drydock and its gate closed, the structure floated at such a draft as to bring its sides above the water level, which permitted of its being pumped out and another caisson built therein while the first one was being finished in a floating position. Not only was great care taken in the mixing and placing of the concrete, but regular tests were made of the ingredients as to secure a water-tight structure.

While the caissons were being built the bottom of the lake where the caissons were to rest was brought to a level by dredging and

sweeping, the sweep being a reinforced concrete beam, 12 by 12 inches by 40 feet long. Since the outer face wall of the caissons was thicker than the rear wall it was necessary to place ballast in the caissons in such a location as to secure a more uniform draft than would otherwise have existed.

After being towed to position the caissons were sunk into position upon the prepared bed by pumping in of water, an operation that took about five and one-half hours, at which time the caissons became safe against any ordinary storm, their tops being but 2 feet above water level. The



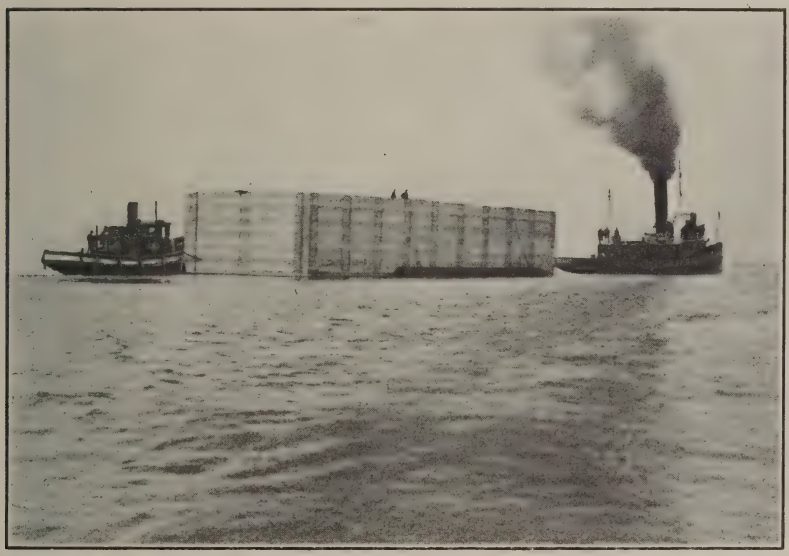
Concrete Crib in position on "Rock" Bottom of Harbor (Western Canada Flour Mills Co., Goderich, Canada).

various compartments were next filled with stone, sand and gravel. A concrete superstructure, placed en masse in situ was then built upon top of the sunken caissons to a height of about 11 feet above lake level, the concrete extending down some 6 feet into the two outer rows of compartments.

During the season of 1913, 600 feet more was added to the breakwater by the same mode of construction as above described, the caissons being 28 feet deep, not 26 feet, due to a greater depth of water. Another 400 feet was added in the season of 1914. The four caissons which formed this addition had been built in the

fall of 1913 and sunk in the harbor during the winter, and were refloated and put into position in the breakwater in the spring, as above described. The total length of this piece of work is 1,660 feet, the longest individual piece of work of its special mode of construction so far built in America, thus showing the great faith the Canadian Government has in Colonel Judson's system. A heavy riprap fill of triangle shape exists along the outer face of the breakwater to a depth of 14 feet.

The exterior walls of the caissons are 16 inches thick, the inside face walls 14 inches thick, all built without taper. The base is

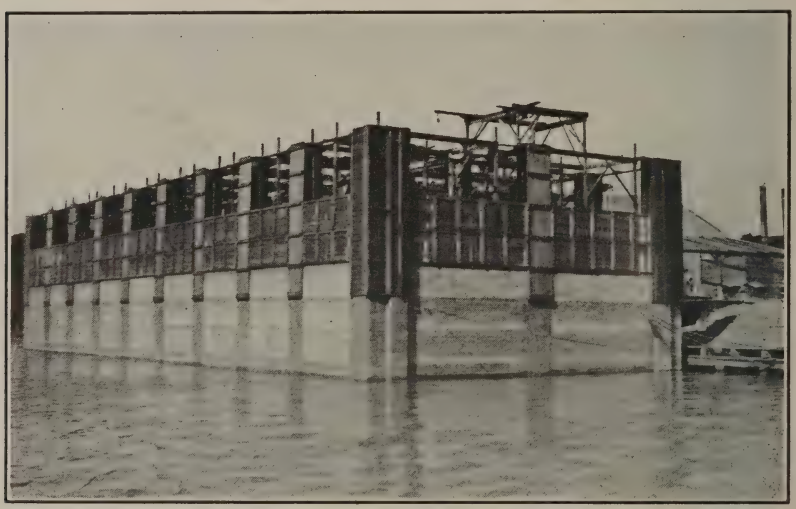


Welland Ship Canal. Towing completed caisson, ready to be sunk, to canal site. These cribs are 111 feet long, 38 feet wide and 34 feet deep. Weight about 2,000 tons. They float when complete with about 16 feet draft. The walls are built heavier at the bottom than at the top of the cribs, throwing center of gravity low, but when completed and afloat they are very sensitive, the center of gravity being near the water surface.

about 24 inches thick, the entire structure being well reinforced and having substantial fillets at intersections of all walls.

In the rebuilding of the Welland Canal a very extensive use of floating concrete caissons is being employed, special new features having been worked out to suit local conditions. This undertaking represents the latest development of Mr. Tromanhauser's system, combined with a system of removable wooden bottoms for

the caissons as worked out by Mr. J. L. Weller, engineer in charge of said improvements, so as to permit of their repeated use and to allow of the interior fill resting directly upon the soil, not on the wooden bottoms, a full description of which undertaking will, no doubt, be given the engineering profession by Mr. Weller when the work has been completed. The caissons being used are 111 feet long, 38 feet wide and 34 feet deep, having eighteen compartments each, weighing about 2,000 tons. When afloat they have a draft of about 16 feet and a freeboard of 18 feet. In order to keep the center of gravity as low as possible, the walls are made thicker at the bottom of the caisson than at the tops. Even with this precaution, it is said that the center of gravity of the com-



Welland Ship Canal. Showing crib nearing completion. The cribs have eighteen compartments.

plete caisson when afloat is near the water line and that they are very sensitive in stability.*

UNITED STATES.

The first application that Colonel Judson seems to have made of

*Since the above was written contract has been been let for the construction of a concrete pier and wall at Victoria, Vancouver Island, Canada, wherein an extensive use of concrete caissons is to be made. The caissons are to be built in a floating dry dock and then put afloat, as done in other foreign ports. Full information of the undertaking will, no doubt, be furnished the engineering profession by the engineer in charge when the work is well under way or completed.

his idea of floating concrete caissons was in the construction of Government breakwaters on Lake Michigan in 1908, these undertakings apparently being the first application of floating concrete caissons in the United States, if not in America.

To determine the effects of frost upon the interior fill of a reinforced concrete caisson, a small structure (7 by 6 by 5½ feet, approximately) with walls from 4 inches to 5 inches thick, was constructed by Colonel Judson in 1907, filled with sand and covered over with a concrete slab and allowed to remain in an exposed position all winter. The results of this experiment were such as to justify the use of concrete caissons on a larger scale in the Algoma, Manitowoc and Milwaukee breakwaters, as no damage was done to the experimental caisson on account of frost action.

Algoma. The twenty caissons used in building the Algoma breakwater* were 24 feet long, 15 feet wide, 12 feet 4 inches high, of a rectangular shape, the caisson part of the breakwater being 511 feet long. As the caissons have to withstand a severe wave action due to the 140-mile fetch of the seas, a heavy construction was used, the exterior walls being 12 inches thick, the athwart middle partition 10 inches thick, while the bottoms are 16 inches thick. The middle partition wall divides the caissons into two compartments of about 10½ by 13 feet, a 1-2-4 mixture being used throughout. Each caisson weighed 119 tons, corresponding to a draft of 9.7 feet, giving a freeboard of 2.6 feet when afloat. After launching, the caissons were towed 12 miles through the open water at a speed of 2 miles per hour to their final place of location, the place of manufacturing having been at Kewaunee, 12 miles south of Algoma. After being sunk by water ballast, the interiors of the caissons were filled with broken stone to within 4 feet of their tops and the rest of the distance, viz, 4 feet, with a lean concrete. A concrete superstructure was erected over the top of the caissons. The caissons rest upon a pile foundation filled in with riprap.

Manitowoc. The building of the Manitowoc breakwater followed immediately after the Algoma, the same plant, etc., being used to construct the twenty-four concrete caissons needed. Twenty-two of the caissons used at Manitowoc were 24 feet long, 14 feet wide and 11 feet 4 inches high. Their external walls are 10 inches thick, their cross walls 8 inches, the bottom 14 inches. The entire set was launched in the fall and kept afloat all winter, no damage

*For views of Algoma breakwater construction see following article.

being done to them through stress of weather. The caissons had to be towed 29 miles in the open lake and were sunk into position the next spring following their manufacture. Two additional caissons of a special design were also used in this work, the entire length of the caisson breakwater being 580 feet. The interior of the caissons were filled with rock to a depth of 2 feet, over which was laid a meager concrete fill for the rest of their depth. These caissons rest upon a rubble fill.

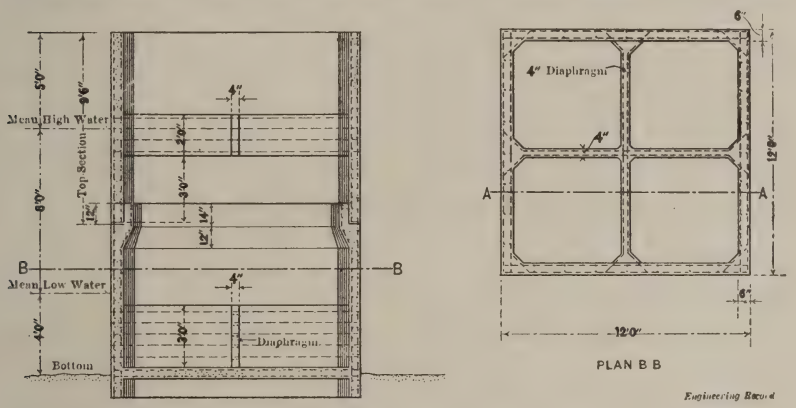
Milwaukee. The next use of concrete caissons by Colonel Judson was at Milwaukee in 1910, the work being consecutive with the Manitowoc undertaking. In all, fourteen caissons were used at Milwaukee. The outermost four are 54 feet long, 18 feet wide and 15 feet deep. Their exterior walls are 14 inches thick; the two interior partitions 12 inches and their bases 18 inches. The cross walls divided the caissons into three compartments about 16 by 16 feet. Each caisson weighed 352 tons, corresponding to a draft of about 13 feet. The inner caissons were ten in number, each one being 36 feet long, 15 feet wide, 12 feet deep. The walls were 12 inches thick, the interior partitions 10 inches and the base 16 inches. Each caisson had three compartments of about 12 by 13 feet and weighed 175 tons. After being sunk onto a pile foundation they were filled with stone, capped with 4 feet of concrete and form a pier 576 feet long.

In building the Algoma and Manitowoc caissons, a pile foundation of nine piles properly capped was prepared and covered with 4-inch planks into which were driven 8-inch wire spikes. These spikes projected into the base concrete and thus bound the planks to the caissons, the planks becoming a part and parcel of the caisson structure when put afloat. The caissons were built on a level keel and not on an incline, as is the usual practice in ship work. In order to prevent any inflow of the water during launching and towing operations, a water-tight wooden deck was secured to the top of each caisson. After the caissons had been completed their weight was transferred from the pile structure to the launching ways and then launched overboard.

In a launching operation at Kewaunee three sets of ways were used, each set being 24 feet wide, 103 feet long with a draft of 6 feet at the out-shore end giving a drop of 3.7 feet. The launching weight of the Algoma caissons were taken as 2.65 per square foot. The piles under the launching ways were figured for 15 tons each.

Glen Cove. The next use of floating concrete caissons in marine work in the United States appears to have been at Glen Cove, Long Island Sound, in the construction of a small wharf for pleasure purposes in the winter of 1909 and 1910; floating caissons were made use of for the piers that support the overhead bridge or runway. Some of the piers were 12 feet by 12 feet in section, others 8 feet by 12 feet. In building the larger size piers, some of which extend 17 feet above the bottom, two caissons were used, one superimposed on the other.

The lower caissons were 9 feet 6 inches in height with a 6-inch bottom of concrete placed at such a height as respects the side walls as to leave a cutting edge, as it were, of 12 inches in depth. The side walls of the caisson were also 6 inches thick. The bottom



Plan and Section of Reinforced Concrete Caisson for the Glen Cove Pier.*

slab is stiffened by two concrete beams 4 inches wide, 30 inches deep, placed at right angles on the center lines of the caissons. The upper rim of the caissons was built with an internal offset of about 6 inches, so as to permit the upper section of the completed pier to fit down over the lower section and thus be held rigidly in place. The offset was 12 inches in depth.

The upper caissons were also 9 feet 6 inches in depth, lapping 1 foot onto the lower caissons, thus making the total height of the entire concrete pier 18 feet, the cutting edges protruding 1 foot into the bottom. The upper caissons were built without a bottom, but have internal bracing beams or struts 4 inches in width, 24 inches in depth.

* Reproduced by permission from *Engineering Record* of Aug. 20, 1910.

The smaller caissons, viz, 8 by 12, have but one internal partition 4 inches wide, 24 inches deep, with walls 6 inches thick. The walls and bottoms of the caissons were reinforced on both sides, as is necessary in this class of work. Special attention was also given to the cement and its chemical properties. All the caissons were built on shore, put afloat and placed in position by filling the interior with ballast. The upper parts were set in place by mechanical power, as having no bottom they possessed no floating capacity.

Huntington Bay, N. Y. During the summer of 1910 resource was had to a floating concrete caisson in constructing the foundation of a lighthouse built by the Government at the entrance to said harbor. The caisson measured 26 by 30 feet, being 16 feet in height. It was divided into four compartments by means of two interior walls. The exterior walls were 16 inches thick, the interior walls 12 inches thick, heavy fillets being worked at the intersections of the various walls and bottom slab. Both surfaces of the walls of the entire structure were reinforced according to the hydraulic pressure they had to withstand. The vertical bars had the same spacing throughout. A 1-2-4 concrete was used.

In the construction of the caisson on the shores of the bay proper blocking was so placed as to be under the outside walls, as well as under the longitudinal center line partition. Eight by twelve timbers were then placed upon the blocking in a level position with a 4-inch wooden flooring worked on top thereof, a wooden sub-base, as it were, thus being formed to the caisson itself. To secure the timber structure to the concrete caisson large lag screws were screwed part ways into the 4-inch flooring, 2 feet on centers across the structure, 1 foot on centers longitudinally. The concrete bottom and the walls were then constructed with the proper reinforcement on the timber platform in the usual manner without any interruption in the pouring, thus resulting in a monolithic structure. After the forms had been removed, the exterior walls were given a heavy grout coating.

Due to the small excess of buoyancy when afloat, the caisson was built in an upright position. After sufficient curing the caisson was launched overboard by means of the usual launching ways and later on towed about ½ mile to its final resting place, where it was sunk onto a prepared bottom. Since the completed structure drew 1½ feet more than the depth of water over its final location on the bar, it was necessary to resort to several means

to decrease its draft so as to permit of it being properly placed. This was done by pumping air into the pockets formed by the 8 by 12 timbers on the underside of the timber base and partly by bodily raising the structure by a derrick, the caisson then being dragged into position by means of tugs. The four interior pockets were then partly filled with water to act as water ballast until the placing of the interior concrete fill. Riprap was also placed around the outside of the caisson immediately after setting to protect it against storms. Afterwards the interior pockets were filled with a 1-3-6 concrete placed in the dry, the work being carried on in two pockets simultaneously and alternately to a depth of 5 feet at a lift. The concrete superstructure of suitable design was then erected. The weight of the caisson was about 315 tons, 290 being of concrete, 25 of steel reinforcement.

GENERAL CONSIDERATIONS.

Before concluding this article, it is fitting to point out a few facts as recommended by those best experienced in the use of concrete floating caissons.

Since the caissons are apt to plunge low in launching unless long and expensive ways are used, the maximum stresses are usually experienced during launching and can be so considered in designing the structure.

While it is possible to launch concrete caissons ten days after their manufacture in fresh water work, it is perhaps better to wait a much longer period when the caissons are to be subjected to sea water action to ensure their absolute permanency.

Concrete caissons possess great stiffness and substantial strength, which permits of them being towed long distances in rough water without fear of danger.

The constructing of concrete caissons at a more or less permanent yard permits of the installation of a suitable plant and the organization of a good working force, both of which will tend to materially reduce the cost of manufacturing and launching the caissons and hence the total cost of the proposed structure.

While in small caissons it is best to have them mono-cellular, in order to decrease their dead weight without making the side walls too thin and hence provide sufficient amount of buoyancy, in large caisson work it is best to have numerous chambers or compartments, since the cross walls can be made of substantial thickness without any appreciable effect upon the buoyancy.

Still, "It is not believed that in many cases it would be true economy to use caissons which would be lighter in weight than conditions of buoyancy requires."

Since it is possible to quickly sink the caissons to position by means of water ballast, they become safe against moderate wave action in a very short space of time after arriving at their final resting place. When sunk the tops of the caissons project above water, thus permitting of the compartments being pumped out in rotation and the interior fill placed in the dry. This affords a cheaper and more substantial system of permanent subaqua construction than any other mode.

With the cost of timber slowly advancing and temporary structures becoming things of the past except in undeveloped countries, permanent concrete construction is rapidly advancing to the front. While "There is probably much to be learned about reinforced concrete floating caissons, and much educational work to be done among contractors, apparently in no other field (of engineering) will reinforced concrete make greater strides in the immediate future than in such works as breakwaters, piers, dock-fronts, revetments, bridge piers and similar structures where reinforced concrete caissons may be employed."

CONCLUSION.

From the great advantage to be derived from floating concrete caissons in harbor development, it would seem that this branch of concrete engineering will find a wide field of applicability now that the use of concrete in sea water is more thoroughly understood by American engineers. In view of the success obtained in foreign countries and the wider use of the system being yearly had, it would seem that with an unrestricted use of said system in the United States upon the expiration of Colonel Judson's patents, the use of floating concrete caissons will be more widely adopted by United States engineers, for if they have been successfully used in deep foundation work why not in hydraulic engineering?

NOTE. The authors desire to acknowledge their indebtedness to numerous articles prepared by builders of floating concrete caissons, from which publications have been assembled the above facts and data. The most thorough and comprehensive discussion of the art that the authors found among engineering literature was an article prepared by W. V. Judson, Lieutenant Colonel, Corps of Engineers, U. S. Army, which appeared in the *Transactions of the Western Society of Engineers* in 1909. The authors' discussion of the caissons built in connection with the breakwaters on Lake Michigan and of the conclusions drawn therefrom as to the advantages, etc., of floating caissons were abstracted from said article of Colonel Judson. The authors are especially indebted to Mr. J. H. Tromanhauser for information furnished covering Cana-

dian experience with floating concrete caissons and for the large number of photographs furnished them covering his work and which are reproduced in this article. As respects foreign practice, the most extensive discussion so far given to the engineering profession seems to have been two articles which appeared in *De Ingenieur*, in 1906 and 1907, written respectively by Mr. S. J. Rutgers and Mr. H. A. Van Yesselsteyn, both of Rotterdam. Briefer discussions as found in *Engineering News*, *Engineering Record*; *Engineer*, London; *Concrete and Constructional Engineer*, *Transactions of International Congress of Navigation*, etc., have likewise been reviewed in the preparation of the authors' article, to all and each of which they acknowledge their indebtedness. While they have tried to include each and every instance of the use of Floating Concrete Caissons in the United States, it is perfectly possible that many such have been constructed of which no account has appeared in public print and hence has not been available to the authors in their researches along the art. Any who may be in a position to furnish data upon such cases would no doubt be adding valuable information to the engineering profession along these lines.

Discussion.

Lieut. Col. W. V. JUDSON,
Corps of Engineers, U. S. Army.

Mr. Taft mentions only such use of caissons for United States piers and breakwaters as was reported in my paper upon "concrete steel caissons," published in the *Journal of the Western Society of Engineers* in 1909.* Since then they have been employed by the United States for breakwaters at Racine and Sheboygan harbors.

Other uses of concrete-steel caissons by the United States, not mentioned by Mr. Taft, are for lighthouse structures on the Panama Canal zone, including the main light at Colon (caisson 45 feet square in plan); for several lights on Lake Michigan; for the Brandy Shoal Light Station (caisson circular in plan and 35 feet in diameter), and for four aids to navigation in Livingston Channel, Detroit River (each caisson 22 by 35 feet in plan).

The Canadian government is employing concrete-steel caissons in the construction of a harbor at Goderich. The Canadian engineer in charge has sent me a print of the cross-section of this breakwater, and I forward herewith for PROFESSIONAL MEMOIRS a tracing of it. (Fig. 5.)

A common difference between the caissons built abroad and those built in this country lies in the employment here of heavier walls and bottoms, the intention being to make the caissons as substantial as the required condition of buoyancy permits. It is readily seen, where the expense of "forming" is relatively so great, that not much is saved in cost by a lessening of the thickness of walls, bottoms and partitions. In fact, the cost might be increased by the greater number of partitions and the larger amount of steel that would incidentally be required. Thick walls give

* The article by Mr. Taft, as printed in this number of PROFESSIONAL MEMOIRS, contains additions made since the above was written.—ED.

greater protection to the steel, permit of the making of better concrete, and result in a more substantial structure.

Like Mr. Taft, I believe that concrete-steel caissons will gradually come into much wider use; and I have no doubt that some day (not coincident with or due to the expiration of my patents, but, very likely, long after that event) there will result great improvement and great saving in many hydraulic structures from the employment of them. Meantime, I am rather disposed to think that the patent, which was taken out in the United States only, helped bring the matter to the attention of foreign engineers who have since used concrete caissons, for the matter of the issue of the United States patent received attention in many foreign technical

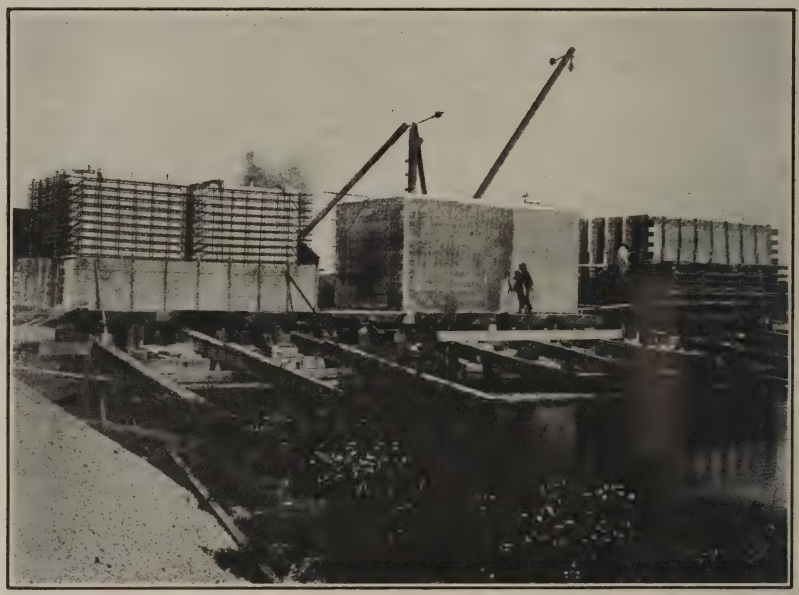


Fig. 1. Reinforced Concrete Caissons under construction, and ready to be launched.

publications; and that the existence of the patent together with the study I and others have given the matter has permitted and facilitated such use of concrete-steel caissons as there has been in the United States.

If I were able to notice a considerable more rapid development of the use of caissons outside the United States than in it, I should be tempted to ask myself whether my patent was, as Mr. Taft hints, of a nature obstructive to progress, and if I came to the conclusion that it was, I should hastily surrender it.

I am afraid, however, that such slow progress is due rather to the following circumstances:

(1) Engineers have been hesitating very generally to use reinforced concrete in sea water.

(2) In fresh water timber always covered is everlasting, and in some cases timber is still cheaper than concrete.

(3) For breakwaters caissons can not compete with rubble-mound work where stone is cheap.

(4) The use of caissons makes necessary a great deal of careful and laborious preliminary study and work in the office, whereas the pigeon holes are full of plans for other kinds of structure.

(5) While a concrete caisson pier or breakwater may in the end be cheaper and better in a given locality than one of another type, yet for caisson work the initial expense and trouble for special plant are great, and this fact acts as a deterrent.

(6) Finally, there is the general objection to adopting anything new. Civil engineering is largely a matter of following pre-

Fig. 2. Reinforced Concrete Caisson for Algoma Harbor, being launched.

cedents, and perhaps there are not yet enough precedents afforded by existing caisson structures to attract the average engineer to the use of them.

I shall not dwell further upon the thought expressed by Mr. Taft that "with an unrestricted use of said system in the United States upon the expiration of Colonel Judson's patents, the use of floating concrete caissons will be more widely adopted by United States engineers," except to remark that my principal purpose in securing a patent was to protect the United States in the free use of reinforced caissons. Since the 1901 patent issued I have had ample proof that if I had not patented the caissons, another would; and the United States, if it had desired to use them, would then have been called upon to pay royalties. The right of the United States freely to use the caissons is carefully provided for.

When the concrete caisson patent was applied for, it is hardly

necessary to say, I had never heard of concrete caissons reinforced or otherwise as having been conceived of, designed or constructed by any other person; and, like Mr. Taft, I have not since heard of any such as having been designed or constructed until some four years after the issue of the patent.

Wooden "cribs" and caissons and steel or iron ones were, I believe, used in this country long before the latter were employed abroad in the works described by Mr. Taft. Indeed, after iron and steel had come to be used for ships, it was not a far thought to use the same materials for caissons, and doubtless many hulks of steel or iron vessels by chance wrecked in suitable locations have filled with sand and served as local breakwaters.

Fig. 3. Reinforced Concrete Caissons at Algoma Harbor—filling and sinking with water, by syphons.

With regard to the "General Considerations," near the end of Mr. Taft's paper, mentioned by me in my article published in 1909, a new "general consideration" occurs to me as the result of observing the surface abrasion from ice action that has occurred at Algoma breakwater. In fresh water in northern latitudes, the caissons should be armored with oak plank from a foot or two below the water upward to the plane of decay; or sheet iron or steel armor should be similarly employed. It would be a simple matter to affix either wood or metal armor when the caisson is cast.

There are enclosed a few photographs* of the Algoma work

* Originally published in Vol. XIV, *Transactions Western Society of Engineers*, and republished here by permission of that society.—Ed.

(Figs. 1-4); cross sections of this work are shown on page 488 of Merriman's *American Civil Engineers' Pocket Book.*

In closing this discussion it may be interesting to refer to the latest developments in caissons used or proposed to be used in Canada and in this country. In Canada they have adopted very large dimensions, while at Chicago Harbor caissons of unusually small dimensions are about to be employed. While local conditions may be responsible for, and justify, the types decided upon in these two instances, their dissimilarity is so striking as to merit attention.

In the Canadian work referred to (Port Weller) caissons, each

Fig. 4. Breakwater at Algoma Harbor—top filling of concrete from mixer on scow—showing manholes in concrete top.

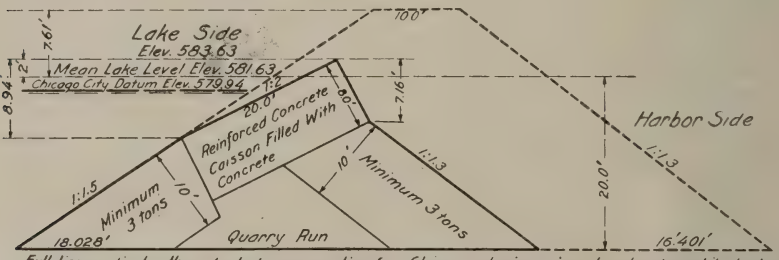
110 feet long, 38 feet wide and 34 feet deep are to be used. An illustrated account of this work is given in *Excavating Engineer* for February, 1915, page 173.

At Chicago Harbor comparatively small, low caissons are to be employed, in a sloping position as shown in Fig. 6 of this article. The caisson here forms but a comparatively small part of the section of the breakwater, but its design and position will materially reduce the cost of the breakwater as compared with the ordinary rubble-mound structure, through the low relief which has been adopted and the narrowness of crest which is practicable. Then,

too, the use of the caisson makes "placed" stone unnecessary, reducing the unit cost below that of a section composed entirely of stone. The caissons when filled will be concrete blocks, each 20 by 8 by 16 feet and of about 200 tons weight.

Other points of importance in connection with the design, pertaining to the cost or stability and durability are:

(a) The practicability of constructing the entire structure by contract. The caissons, empty, would weigh but about 60 tons and can be manufactured on an ordinary wharf and placed in the water after completion without expensive ways and without necessity of floating the caissons before they are completed. All the caissons may be built without moving the concrete plant and in accordance with factory methods, each caisson being moved away from the plant on completion. This insures the maximum of economy in construction.



Full lines indicate the adopted cross section for Chicago sloping caisson breakwater, while broken lines show the rubble-mound section which has been employed upon the Great Lakes in the majority of cases.

Study of Sections
EXTERIOR AND SHORE ARM BREAKWATERS
CHICAGO HARBOR, ILL.
20 ft. depth Dec. 1914.

Fig. 6.

(b) There will be comparatively little reflection of waves from a section of the design shown in Fig. 6. This avoidance of reflection will make conditions better for vessels, during storms, approaching the harbor entrance from the lake, and will obviate the need of converging arms.

(c) While the section is low, it is believed to be high enough. Water passing over will be sufficiently cushioned on the water behind the breakwater to prevent injury to the harbor slope (the latter 10 feet thick, of 3-ton stones). Waves would not be propagated through the breakwater and the water passing over would do no harm to vessels.

From its most distinguishing feature I call the Chicago design a "sloping caisson breakwater." It is expected that work upon it will begin during the approaching working season. Until some time has elapsed after its construction this "sloping caisson" structure, by reason of its novelty, must be regarded as to a certain extent experimental.

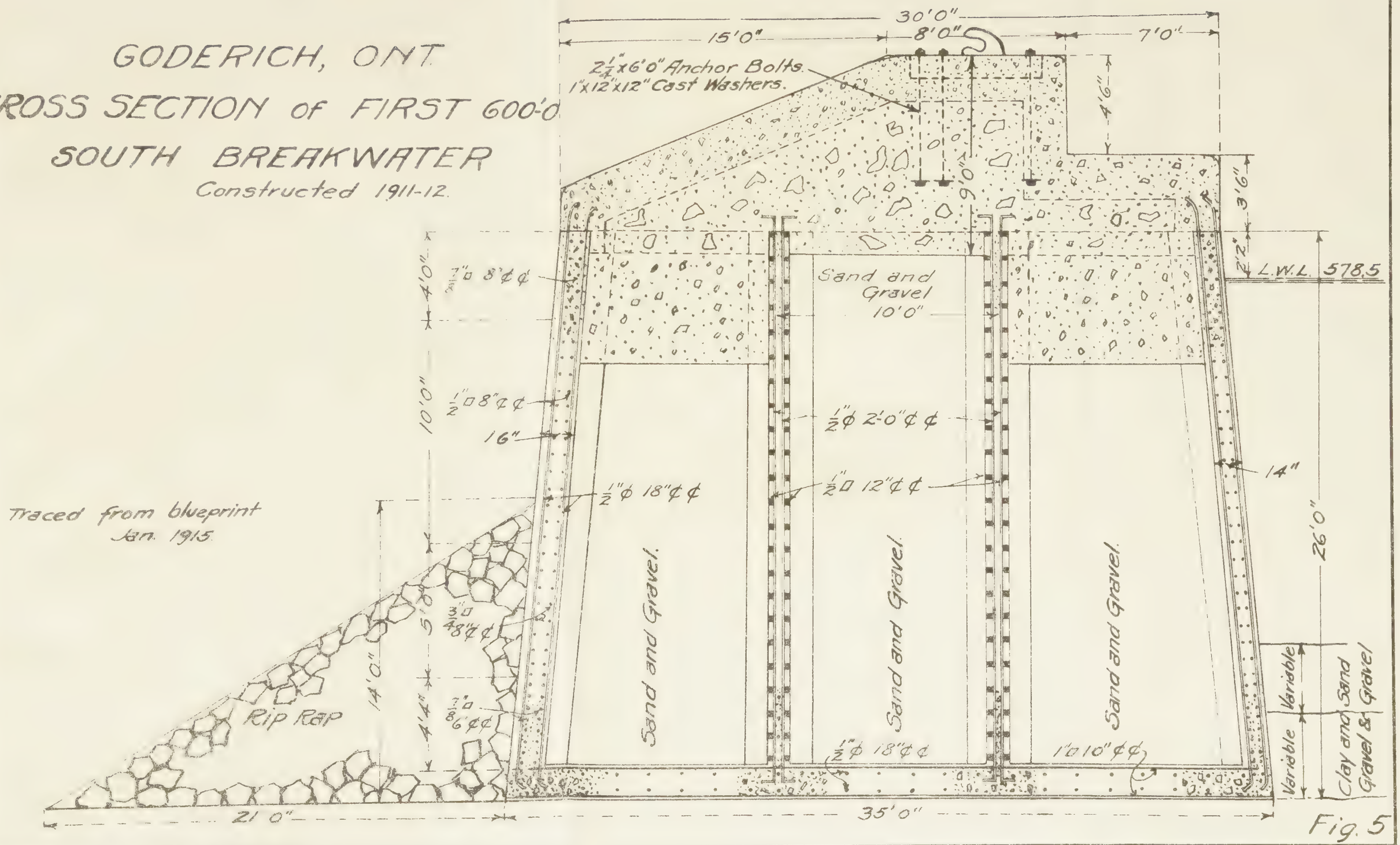
Fig. 5

LIBRARY
OF THE
UNIVERSITY OF ILLINOIS

GODERICH, ONT.

CROSS SECTION of FIRST 600'-0" SOUTH BREAKWATER

Constructed 1911-12.



Traced from blueprint
Jan. 1915

Fig. 5

The Effect of Levees on the Flow of Sediment in Rivers.

(A Discussion of the Article on "The Huai River Conservancy Project.")

BY

Col. C. McD. Townsend,
Corps of Engineers, U. S. A.

The discussion of the Yellow River by the Board of Engineers, which is printed in the paper on the Huai River Conservancy Project in the January-February number of the Professional Memoirs, has not only an important bearing on that project, but can be construed by those unfamiliar with the principles of river hydraulics as condemnatory of levee construction on other rivers. The following analysis of the paper is therefore submitted:

On page 119, it is stated that "In its confined channel, between primitive dikes, the Yellow River has been building up a restricted delta and endeavoring to create a gradient commensurate with its needs." And in proof of this assertion Plates II and III are submitted to show both a raising of the bed and of the limited flood plain between the dikes. The force that produced these results must have been exerted between the dikes, and if the fill was caused by a confined flood discharge, such a discharge must have flowed through the cross-sections shown in Plates II and III.

The paper contains very little physical data relative to the river, but in the quotation from Mr. Elias is described a flow having a width of 12 miles. In the Mississippi delta such a width of flow would be accompanied by a mean depth of about 6 feet and a mean velocity of about 3 feet per second, and is indicative of a flood discharge of the Yellow River of over 1,000,000 second-feet. The Mississippi River Commission moreover has data which confirms this assumption. In the first cross-section (Mengkung), shown on Plate II, the area of the cross-section to the tops of the dikes is about 8,000 square feet, and it would be necessary for the water to flow with a mean velocity of over 125 feet per second for

1,000,000 second-feet to pass through such a section, a rather violent assumption when it is considered that for 2,300 feet of the cross-section the water would have a depth of less than 5 feet.

The maximum mean velocity observed on the Mississippi or Missouri rivers during floods is 12 feet, which occurred through the Merchants Bridge, at St. Louis, and was sufficient to scour to the underlying rock. Except in this restrained section the maximum observed velocity is 10 feet, and such a velocity is sufficient to produce a scour in bends such as the Mengkung cross-section indicates exist at that locality.

A cross-section which would permit a flood discharge, with a velocity of 10 feet per second, would have an area of over 100,000 square feet. The normal flood cross-section of the Lower Mississippi River varies from 180,000 to 225,000 square feet.

It is therefore evident that the deposits shown on the plates did not occur when a confined flood was flowing between the dikes, nor is the cause of the fill difficult to determine. In the quotation from Tyler, it is stated that in 1853 the Hwang Ho broke its dike and flowed into the Ta-Ch'ing-ho, just as in 1882 the Mississippi River started to break into the Atchafalaya River, but instead of restricting the flow, as the Mississippi River Commission did, by the construction of sills at Simsport, the Chinese allowed the break to enlarge, and as a result the crevasse became the main outlet of the river. It is a general law of river hydraulics, which has repeatedly been emphasized by the Mississippi River Commission in its reports, that if a river has two outlets there is a tendency of one or both of them to fill. For the last thirty years it has been necessary constantly to dredge the mouth of Old River to maintain a navigable channel into the Red and Atchafalaya rivers.

The conditions on the Old Yellow River are similar to those that occurred on the Colorado River, when a crevasse turned its waters into the Imperial Valley. In less than one year the old bed of the river silted up so that there was no flow through it except at flood stages. The old bed of the Yellow River has had some sixty years in which to produce similar results.

Through the courtesy of Mr. J. A. Ockerson, member of the Mississippi River Commission, photographs of the old bed of the Colorado, just below the point of diversion, are given. These photographs were taken about a year after the crevasse and show the camp of the survey party in the old bed, a survey party at work in the old bed, and upper end of old bed.

It was not levee construction, but the failure of levees that caused the fill in the Old Yellow River, and if the Mississippi River should be diverted into the Atchafalaya, a similar fill would occur near New Orleans.

With the exception of these cross-sections, the paper consists principally of extracts from reports of travellers, which afford strong presumptive evidence that the four thousand years that the Yellow River has been leveed, have not produced any such rise of the river bed as should cause apprehension to the advocates of a levee system on the Mississippi River.

Tyler states that in 1853 the bed of the Ta-Ch'ing-ho was 1 "li"

Fig. 1. Surveying dry bed of Colorado River, Sonora, Mexico, September, 1910.

wide and was sunk in the plain to a depth of some 50 feet or so. What caused such an excavation in the plain of China? It corresponds very closely in depth to the numerous lakes which are found in the delta of the Mississippi and which were once the bed of that stream, though the width—one-third of a mile—is somewhat less, and it is a natural assumption, particularly after glancing at Plate I, that the Yellow River some time previously in its wanderings caused the excavation, and is therefore capable at the present time of repeating the same action.

In 1903, after the Hwang Ho had been leveed, he states that the flood level is about 15 Chinese feet (assumed as the "chih," about

13 inches) above the level of the original plain, a not excessive height when it is considered that in 1912 the flood level of the Mississippi was over 20 feet above the level of its delta, from Cairo to the mouth of Red River.

The levees shown in the various sections are from 10 to 25 feet in height, averaging considerably less than those at present existing on the Mississippi River. If five hundred and thirty years of fill of the old Yellow River, under the abnormal conditions described in the paper, only necessitated the construction of levees of such dimensions, a normal river like the Yangtse or Mississippi can be leveed without any apprehension of undue levee heights.

The slope of the Yellow River—1 foot to the mile—while greater than that of the Yangtse or the Mississippi, is exceeded by many rivers in the United States. The Colorado River, from Yuma to its mouth, has a slope of 1½ feet to the mile. The Missouri River has an average slope of over eight-tenths of a foot and has created a channel through which its sediment flows without raising its bed.

A knowledge of the cross-sections of the existing Yellow River would be most valuable to the discussion. The writer has been informed that two such sections were taken by the Board, and the President of the Red Cross Society courteously placed all the records of the Board at his disposal, but as these records are very voluminous and not indexed, in the limited time he had available he was unable to find them. He did find, however, cross-sections of the Huai River, which had an area over three times that of the Mengkung section with a discharge of about 80,000 second-feet.

There is moreover a significant statement in the complete report of the Board (page 13) that since levees restraining the floods of the existing river have been constructed, "the surface of the river at ordinary stage is below the bed of its ancient channel and that the river above the break does not at present rise to its former flood plane even at times of maximum flood." As the existing river as shown on the map does not differ materially in length from the old river, and its total slope can not therefore have changed, this is strong presumptive evidence of scour from levee construction rather than fill.

Although the Red River has an average slope of but five-tenths of a foot to the mile, a paper presented to the Mississippi River Commission in 1881* gives a description of its flow in the delta of

* "Report on Red and Atchafalaya Rivers," Report of Mississippi River Commission, 1881, page 129.

the Mississippi which closely resembles that of the Yellow River in the delta of the Yangtse. According to this report, the Red originally flowed down the Bayou Teche, then broke into the Black River, and wandered over the Atchafalaya Basin in a manner similar to the Yellow River in the Chinese Plain, until it finally caved into the Mississippi, just as the Yellow River appears at certain periods to have flowed into the Yangtse.

The cause of these vagaries was ascribed to obstructions in the bed of the river caused by snags and the lodgment of débris against them, locally known as "rafts," which is considered more logical

Fig. 2. Engineers' camp in dry bed of Colorado River, Sonora, Mexico, September, 1910.

than the upheaval of the earth's crust, which is given as the cause of the changes in the Yellow River. Since these rafts have been removed, the flow confined to a single channel, and levees have been constructed confining its flood discharge, the Red River has developed a channel which will carry its sediment, and in some stretches there has been a marked lowering of its bed, as explained in a paper on the flow of sediment in the Mississippi River recently presented to the Mississippi River Commission by the writer.*

* A reprint of the paper referred to will appear in the next number of PROFESSIONAL MEMOIRS.—ED.

The enlargement of the Atchafalaya River, which threatened to cause a diversion of the Mississippi River down that stream, was also caused by the removal of "rafts," and the limited flow permitted is filling Lake Chicot and Grand Lake, but where leveed the section is scouring, and it may be noted that if the Mississippi River were to be diverted from any cause into either the Black or Atchafalaya rivers, the levee problem would be a very serious one, as for long distances the levees would have to be built with their bases approximately at low water instead of at a bankfull stage.

The mountains are gradually being degraded and the deltas of rivers are being extended into the sea, so that there is no question but that during geological eons rivers raise their beds, but such changes are of little practical moment, as they do not usually amount to the raising of the bed of one-tenth of an inch a year. The only approximate measure we have of such action is on the Nile. The paper presents no evidence that anything more alarming is occurring in China.

The writer has recently prepared a paper for presentation to the Committee on Floods and Flood Control of the American Society of Civil Engineers, which is so pertinent to the question of levee control of rivers that it is presented to the readers of the MEMOIRS.

Every alluvial stream flowing in a valley which has been formed by its own deposits, builds up its banks to a certain height, which is generally known as the bankfull stage. Below that stage the stream flows in a more or less clearly defined trough; above it, it spreads over the alluvial valley. When an unleveed river rises above the bankfull stage there is therefore an abrupt change in its regimen. The area of its cross-section very rapidly increases, and ceasing to follow the sinuosities which its low-water channel has created, its currents tend to flow across points, materially changing the slope. These changes in regimen have a tendency materially to affect the movement of sediment, causing deposits at certain localities and scour at others.

When the river is leveed this abrupt change in regimen is prevented to a greater or less extent, dependent on the distance of the levee from the river bank. The function of a levee as far as river hydraulics are concerned, is therefore to extend the natural banks up to extreme flood stages and continue and possibly intensify the conditions which exist at a bankfull stage; that is, wherever a stream tends to scour or fill at a bankfull stage, that tendency will be somewhat intensified at extreme flood stages if the stream be controlled by levees which have been properly located with reference to its banks.

In the flow of water from its source in the mountains, to the sea, there are some localities where it has an erosive action; at others it causes a fill. Thus, at the summit of a steep hill there is marked erosive action and the material thus scoured is deposited in a fan-shaped bar in the valley at the foot of the declivity. If a stream flowing down such a hill is leveed, the scour at the top of the hill is localized and intensified, producing a deep gully, while at the entrance to the valley there is not the opportunity for the detritus to spread, and the lighter earthen matter is carried long distances by the confined current, but the heavier stone, gravel and sand are deposited where the slope changes producing a marked raising of the stream bed.

Fig. 3. Upper end of dry bed of Colorado River. Vegetation stands on natural surface of ground beyond old bed. Elevated mesa on left of picture.

A large river differs, however, from a mountain stream, in that there has generally been produced an equilibrium between the forces which produce scour and fill. There is a large amount of sediment moving down the river, but the amount which is discharged equals that which is brought in by its tributaries. In a given bend the section enlarges and contracts as the river discharge increases or diminishes, and due to cutoffs or crevasses there may be local disturbances of the equilibrium. Levees may intensify these local actions, but when a river as a whole is considered they do not increase the deposition but merely tend to accelerate the flow of the sediment.

The statement so frequently reiterated that levees cause a permanent rise in the bed of a river, is not only contrary to the theories of river hydraulics, but has no foundation in fact. It has generally arisen from the correct observation that levees raise the height of the surface of the water during floods and the fallacious assumption that therefore the bottom must have been correspondingly elevated, ignoring the fact that a flood which in a state of nature spreads over an entire valley, by levees is confined to a much smaller width and must raise the water surface to produce the same discharge section, unless the latter can be created by scouring the river bed.

The results of observations on the Mississippi River are summarized as follows:

On the Lower Mississippi there are six stations where the discharge observations have been annually taken. At three of these the same section has been preserved during the entire series of observations, that is, Columbus, Ky., Red River Landing, and Carrollton, La. A comparison of the area of section at the time the maximum discharge was observed during high floods shows that while varying from year to year the areas in recent years exceed those first observed.

A river is constantly changing the area of its cross-section even below a bankfull stage where a levee can exert no influence, and marked changes in cross-section are to be expected at flood stages from causes not connected with levees. A comparison of cross-sections at a few localities is therefore not conclusive of the general tendency of the river. Better results can be obtained from averages of areas over long stretches of the river at different periods. The Mississippi River Commission caused a survey to be made from the mouth of the Arkansas River to Donaldsonville, La., in 1894 and 1895, with this object in view. A comparison of this survey with that of 1881-1882 indicates a tendency to enlarge the average cross-section rather than to diminish it. The mean low-water stages at those gauges which have been established for a period of forty years, also show in general a similar tendency, and particularly at Lake Providence, La., and Vicksburg, Miss. Such observations indicate that a raising of the river bed has not occurred in the last forty years.

Extreme floods rarely occur and during the intermediate years the river is less active; in fact, during low-water years it may tend to counteract any tendencies which would be shown during extreme floods. Positively to demonstrate what is the effect of levees on a river bed will therefore require many years of observation. In a new country, like the United States, the improvement of our rivers has not progressed a sufficient length of time to furnish the requisite data. It is therefore necessary to seek for information from the rivers of Europe. Fortunately, the Rhine and the Po have been leveed for long periods, and on both rivers there is data

available extending over a period of over one hundred years, which has been carefully analyzed by Dutch, German and Italian engineers.

In the leveed portion of the Rhine, below Dusseldorf, Fijnje (*Beschouwingen over eenige rivieren erste gedelte*) made a comparison of the height of the lowest water, which occurred in each ten years, from 1772 to 1880, above the Amsterdam zero at Cologne and Emmerich. These observations indicate a marked lowering of extreme low water in recent years at Emmerich and possibly a slight lowering at Cologne.

Hagen (*Wasserstande in den Preussischen Stromen*) made an examination of the gauge readings at Dusseldorf, from 1800 to 1879, and also at Cologne, from 1846 to 1879. He found at both Dusseldorf and Cologne that the observations indicated a lowering of extreme high water in recent years, a stationary mean stage and a rising of low-water stages averaging about one-half of an inch per year, but there is an uncertainty whether even these slight changes are due to levee construction or river regulation, which was also in progress during the periods under consideration, and his deductions at Cologne from thirty-three years' observations differ materially from those of Fijnje based on one hundred and eight years. Lombardini published several articles on the gauge relations of the Po, and in a paper (*Il Grande Estuaria Adriatico*, 1868), in discussing the mean heights of extreme low water during the two periods, from 1817 to 1850 and 1851 to 1867, at Ostiglia, Sermide, Qualrilli and Ponte Lagoscuro, finds evidences of an increase in the height of mean low water in the latter period amounting to an average of about two one-hundredths of a foot per year.

Other Italian writers have also discussed the subject, and there has been recently published an analysis of the records of the Po at Ponte Lagoscuro, from 1806 to 1907 (*Il Po Nelle Effemiridi Di Un Secolo*, 1913, by G. Fantoli), which is so complete a refutation of the oft-repeated assertion that the bed of the Po has risen from levee construction that it merits special attention.

Of the gauge at Ponte Lagoscuro the author states that it is located below the main tributaries, but sufficiently distant from the sea not to be affected by its action, and is situated on a stretch of river defined and limited by levee lines so that it summarizes most justly the regimen and character of the river.

He discusses the high and low water of each year and a mean stage which he obtains by taking the mean of the crests of the rises for a year. He then computes the means of those stages for periods of ten, twenty and forty years, and also similarly investigates the duration of the discharge below certain assumed heights. There is given in an appended table the results as to mean stages. His conclusions are that a period of ten years is not sufficient for determining general laws; that variations in rainfall and other accidental or unknown causes will not be eliminated in so short a

time, but that if a result slightly indicated in a ten year period is more pronounced in one of twenty years, and still more so in one of forty years, it may logically be inferred that it arises from a fixed cause, and that when all the elements he discusses show the same tendencies though in different degrees, the deduction is most conclusive. While there is an increase in extreme floods, which he attributes to the completion of the levee line, it is shown in the table appended that the mean flood stage during the last forty years of the hundred years under consideration is two centimeters below that of the first forty years, and the mean of the midstage and of the low-water stage is fifteen and twenty-three centimeters below respectively.

In reference to the river section at Ponte Lagoscuro, his analysis demonstrates that while there is an enlargement and diminution of the area as the river rises and falls, it tends to oscillate around a mean which does not materially change during the century under consideration and that the changes observed in the tables bear no appreciable relation to changes in the river bed.

His final conclusion is that the changes observed are principally due to the abstraction of water from the river for purposes of irrigation, but that there is a small decrement inappreciable during floods but evident in low water which is not accounted for by irrigation and may be due to the deforestation of the country during the period under discussion. He also discusses the deductions of Lombardini and others and demonstrates that the slight changes he observed resulted from the short periods that were available for study.

The different deductions of Fijnje and Hagen at Cologne, above alluded to, may be accounted for by the same assumption.

RIVER PO AT PONTE LAGOSCURO.

Means by successive ten and twenty-year periods of the maximum, minimum, and mean heights, in meters.

Ten year periods.	Twenty year periods.	Annual maximum above the zero of the gage.		Annual minimum below the zero of the gage.		Annual mean above a horizontal 6 meters below zero of gage.	
		Mean of 10-year periods.	Mean of 20-year periods	Mean of 10-year periods.	Mean of 20-year periods.	Mean of 10-year periods	Mean of 20-year periods.
1807–1816	1807–1826	1.668	1.394	4.471	4.568	3.519	3.310
1817–1826		1.120		4.666		3.102	
1827–1836	1827–1846	0.865	1.432	4.825	4.586	2.918	3.236
1837–1846		2.000		4.348		3.554	
1847–1856	1847–1866	1.119	1.370	4.488	4.608	3.441	3.260
1857–1866		1.621		4.729		3.079	
1867–1876	1867–1886	1.190	1.313	4.757	4.821	3.076	3.033
1877–1886		1.436		4.886		2.990	
1887–1896	1887–1906	1.258	1.475	4.937	4.795	3.098	3.211
1897–1906		1.693		4.654		3.324	

The Battle of Nashville.

BY

Maj. Wm. W. Harts,
Corps of Engineers, U. S. A.

The battle of Nashville was fought on the 15th and 16th of December, 1864, between the Confederate forces on one side under General Hood, amounting to about 54,000 men, less casualties at Franklin, aggregating about 7,500, and General Thomas on the Federal side, commanding about 55,000 men. Hood's forces before the battle consisted of three infantry corps, amounting to 38,117 men under Cheatham, Stewart and Stephen D. Lee, 12,753 cavalry under Forrest, and 3,068 artillery. The Federal forces consisted of about 46,720 infantry and artillery and 8,239 cavalry. It will thus be seen that although the Federal infantry was considerably stronger than Hood's, the Confederate cavalry, on the other hand, was much superior to that commanded by Thomas.

In entering upon a description of this battle it will be well to review briefly the events which led up to it, and analyze the motives of the two commanders and the purpose sought to be achieved by each. While the Army of Virginia and the Army of the Potomac were engaged in the East in continuous conflict, the fortunes of war now resting on one side and now on the other, events in the West permitted what has been called an enormous flanking movement by which the left flank of the Confederacy was several times turned. The first step in this comprehensive movement was the opening of the Mississippi River in the Vicksburg campaign, which cut off a large territory, comprising Arkansas, Louisiana, and Texas, and prevented it from contributing with any degree of success the men, supplies and money which these States would otherwise have been able to send to the army with which they were allied.

The next step in these large flanking operations occurred about a year later after Chattanooga had been relieved by General Grant from the siege set up by General Bragg and the Confederate army.

At this time General Sherman was enabled to commence his famous flanking movements toward Atlanta, which were contested by Johnston step by step, with great ability. The Confederate army was forced to yield, by overpowering numbers, until finally General Johnston was shut up in Atlanta by Sherman's army, and the Chattanooga situation was thus completely reversed. The fall of Atlanta soon after permitted the commencement of a new campaign.

General Johnston's record shows him to have been a wise and able leader, and results showed the wisdom of his strategy; but the Confederacy, eager for successes that would increase its prestige, encourage recruitment, and perhaps induce foreign intervention, demanded decisive victories. Accordingly, Johnston, whose Fabian policy before Atlanta had been condemned, was relieved by the orders of the president of the Confederacy, and General Hood, a young man thirty-four years of age, whose impetuosity and daring had been well tried on several battlefields, was placed in command. To him was entrusted the important task of preventing Sherman's army from cutting off another area from the Confederacy in the furtherance of the strategy of the Union commanders and in him were reposed the hopes of holding some semblance of control in the West against the constant encroachments of the Union army on the territory from which the South received its supply of men and food.

It was finally the avowed purpose of General Sherman, in inaugurating a new campaign, to establish a new base on the seacoast, leaving Chattanooga to the protection of Thomas's army (then in Nashville), and thereupon to continue the flanking movement on a grand scale by attacking Lee's army in the left and rear, after slicing a second area from the Confederacy, consisting of Alabama and Georgia. His greatest difficulty was believed to lie in shaking himself free from Johnston in his front, but his success in this was soon to come in a way that had not been expected; for Hood, upon receiving command of Johnston's forces, after several ineffectual and costly engagements, left Sherman's front with a view to attacking his communications in the rear. In taking this step his plan was to invade Tennessee and Kentucky, and then, after defeating the Union army at Nashville, to march to the assistance of Lee in Virginia, who was being hard pressed in his defense of Richmond.

It was Sherman's first purpose to follow Hood in case he should

attempt to cross the Tennessee River near the Union line of communications, and he actually did send some troops after him; but Hood, although acting under the approval of General Beauregard, moved farther westward with his entire force and attempted to cross the Tennessee River at Decatur. Here he was thwarted by a small Federal force under General Granger, who persistently denied him the river at this point, causing him considerable losses which he could ill afford. Requiring more troops and supplies, Hood immediately marched westward to Tuscumbia, where troops from Texas and supplies from Mississippi were to reach him through the small town of Cherokee; but the railroad between Cherokee and Tuscumbia had been destroyed, and the supplies had to be hauled over miserable roads, occasioning great delay and much hardship. The troops never appeared.

In the meantime Sherman detached Schofield's command and sent him back to assist Thomas, whose forces at that time did not number more than half of Hood's. It was thus seen to be necessary that the troops in Nashville should be strengthened by every possible means if a successful resistance was to be made. Gen. A. J. Smith, who had been in western Missouri opposing the irrepressible Gen. Sterling Price, was ordered to Nashville post haste. Gen. J. H. Wilson, who had been endeavoring to stop the irritating cavalry raids of General Forrest, was likewise recalled, and all the troops in Tennessee and Kentucky that were available were hurried to General Thomas in order to increase his force.

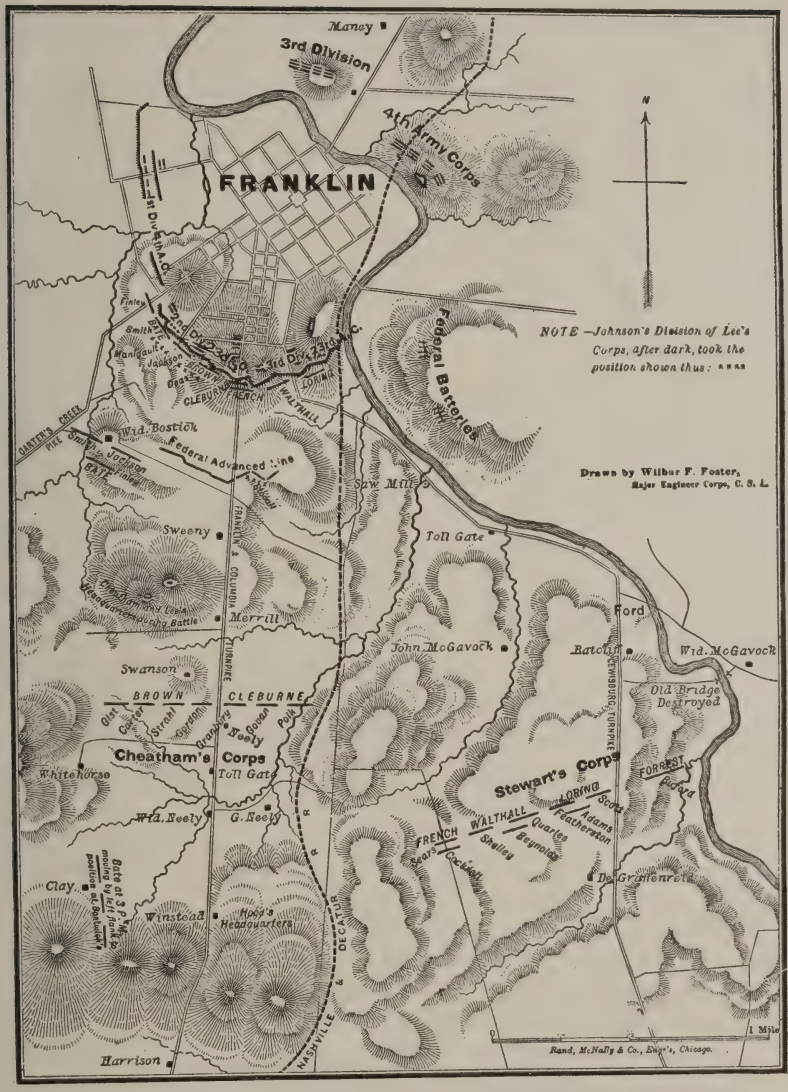
As above mentioned, Hood was expecting fresh troops from Texas, but these failed to arrive, and so in November he crossed the Tennessee River at Florence without them and commenced his movement northward. All his army was filled with enthusiasm. This was the long-heralded beginning of the campaign in Tennessee, which promised so much but which was to end so disastrously at the battle of Nashville.

Schofield had been sent south to Pulaski to observe and retard Hood's approach, but was forced to withdraw northward by a flank movement of Forrest's which almost cut him off from connection with his rear. He then fell back on Columbia, where there was a small fight. At Spring Hill, about midway between Columbia and Franklin, several roads cross, and it was necessary that Schofield should possess this strategic point before his adversary could reach it, otherwise he could not expect to make his safe return to Nashville possible. Forrest was also moving for this point, and both Cheat-

ham and Cleburne were putting forth every effort to cut off Schofield's retreat. By a narrow margin, however, General Updike of the Union army secured this point just before the arrival of General Forrest, and held it against the Confederate attack. He thus protected General Schofield's line of retreat and saved the Union army from peril. Hood saw that his enemy had extricated himself from the trap he was laying for him, for it was his purpose to defeat Schofield if possible at Spring Hill, and in any case before he could connect with Thomas's force. Hood therefore blamed his generals, particularly Cheatham and Cleburne, for disobedience of orders; thus causing many heartburnings and deep feelings of resentfulness. He recommended that General Cleburne be relieved by another officer, but finally withdrew this message.

Schofield was now in full retreat toward Nashville and in order that his prey should not escape him at the next point of contact, Hood, hoping to crush him completely, hurried his forces against Schofield while the latter was endeavoring to retreat across the Harpeth River at the little town of Franklin. Here one of the most desperate battles of the war resulted; for although its duration was but a few hours, the number of killed and wounded on both sides was appalling, and nowhere during the war was there fiercer fighting. Hood's force was far superior to Schofield's, and the latter was encumbered with over 800 wagons. General Wagner, of the Union forces, had been directed to hold a small knoll in advance of the regular lines, which extended for the distance of about a mile around the town of Franklin. Under his instructions on no conditions was he to give battle, but was to yield to pressure. The Union lines in front of the town consisted of the brigades of Cox, Ruger, and Kimball, extending from left to right. General Wood of Schofield's army occupied the heights on the north side of the stream, and General J. H. Wilson was protecting the Federal left with his cavalry against Forrest, who was seeking Schofield's flank and rear on the north side of the Harpeth. Opposing Schofield's lines were the three corps of Hood's army, commanded by General Stewart on the right, General Cheatham in the center, and General Bate on the left, respectively; Cheatham's Corps being composed of Cleburne's and Brown's divisions.

The Confederate generals, still smarting with the stinging rebuke of Hood, after the Spring Hill disappointment forced the fighting with a desperateness that has seldom been equaled. The advanced position of Wagner was charged with an impetuosity



MAP OF THE BATTLE-FIELD OF FRANKLIN, TENNESSEE. FROM THE "BIVOUAC" FOR JUNE, 1885.

that carried everything before it; and, in fact, his retreat was so precipitate that the troops that had improvised intrenchments around Franklin could not fire for fear of injuring their own men, so that the rush broke down Schofield's lines and carried the Confederates through, enabling them to capture several guns and many prisoners, almost without firing a shot. This delayed the attack somewhat, and during this short delay the Union lines were reformed back of some of the first defenses; the first position was then recovered, and at this point charge after charge was repulsed; the dead lying sometimes five and six deep in rows where they fell. This battle had a very important effect on Hood's campaign. He found that his advance would be resisted with a determination that he had scarcely taken into consideration, and his losses seriously crippled him.

The loss on the Federal side at the battle of Franklin was 2,326, of whom 189 were killed; whereas that on the Confederate side was nearly three times as great (6,252, of whom 1,750 were killed). Among the killed were several of Hood's best generals.

Schofield reached Nashville the next day, pursued by Hood, whose army had been shaken in morale and hurt in strength by the check it had sustained a few days before at Franklin. Nashville had been well fortified during the months preceding, by a chain of redoubts connected with rifle pits, extending from the river near the penitentiary along a ridge of hills to the hill on which is now Belmont College. Continuing, the line ran to the reservoir, then through Fort Negley, and thence to the river east of the city following the high ground. Hood on coming up assumed a position on a parallel ridge of hills extending from the Hillsboro pike on his left to the Chattanooga railway on his right. Montgomery Hill was his most prominent point. Cheatham was given the right flank; Stephen D. Lee, whose troops had not borne much of the brunt of the fighting at Franklin, was placed in the center, and Stewart was located on the left. It was Hood's intention to fortify a number of hills westward, extending his line so as to protect his left flank. Other works similarly protected his right flank as far as the railroad. About this time peremptory orders were sent to Thomas to attack, but he was not quite ready.

Hood's position was now a very precarious one. He realized his inability to attack the fortified position in his front with any hope of success, for he had been informed of the large increase of

numbers in the Federal Army, and knew from his experience at Franklin that such an attack would likely be hopeless. The river behind Nashville was patrolled by gunboats; both flanks of the Federal army were well protected on the river, and nothing but a frontal assault seemed possible. This Hood realized would be suicidal, in view of his previous serious losses and the discontented condition of his army. His reinforcements had failed him; his ammunition and supplies were low; the necessity of an advance of some sort was urgent, for a retreat would be a confession of weakness that would throw away all he had gained. Notwithstanding all this, he was believed to be preparing to attack the city, when all military operations were delayed by a storm of several days, which covered the country with ice and sleet.

Between the time of the battle of Franklin, on November 30, and the 8th of December, Thomas had received many additions to his command, practically doubling his force. These, however, were of such heterogeneous units that it was a motley array he had behind his defenses. His lines were composed of dismounted cavalry, recent recruits, quartermaster's employees, and negro troops, as well as seasoned soldiers. In his disposition of them he placed Steedman's corps on the left, resting on the river; Schofield on Steedman's right, Wood in the center, and A. J. Smith, who had recently come from Missouri, on the extreme right. Wilson, with his cavalry, was first in Steedman's rear, but later in Smith's.

It seems to have been Hood's intention to make an attack on the Federal right, and Chalmers's cavalry was sent along the Charlotte pike to open the action; although some think he was contemplating a new movement completely around Nashville and into Kentucky. Between the 8th and 13th of December a severe storm of rain, sleet, and snow took place. No movements of any sort were practicable, owing to the ice, and both armies lay idle. It was at this time that thoughts of relieving Thomas were prominent in the minds of Grant and others in authority in Washington. His delay, his refusal to attack when ordered, and his lack of full explanation filled them with apprehension, and orders were issued relieving him and putting Gen. John D. Logan in command. General Grant on the 15th reached Washington on his way to take command himself.

On the 14th, however, there came a warm rain causing a thaw of the ice, which all recognized was the signal for important

movements of some sort. The rain at the same time hampered the movements of the troops by softening the ground. On the morning of the 15th of December the country was enveloped in a dense fog, and under cover of this fog the movement of the Federal Army was commenced. General Thomas's plan of battle was to make a feint attack on Hood's right (located near the present fair grounds), and lead him to believe that the main attack was to be upon this part of his line. At the same time one of Wilson's cavalry divisions was sent along the Charlotte pike on the extreme west of the town, to drive back Chalmers's cavalry. Chalmers made but slight resistance, and was brushed away with but little fighting.

Thomas's next plan was to create pressure upon the Confederate right and center, in order to hold his troops there, and then to swing his own right in a long flanking movement and exert crushing force upon the Confederate extreme left. The great wisdom of this action is seen when we realize that with Steedman's division and Wood's corps Thomas held practically Hood's entire army in their trenches while he completed his flanking movement with the greater part of his own forces. A. J. Smith on the extreme right was ordered forward, and as soon as the roads were cleared Wilson with his cavalry was pushed still further on his own right and against Hood's left flank. Schofield's Corps was at first held in reserve, having had hard fighting at Franklin, but was used later for driving home the crushing flank attack. It came up as soon as Wilson's cavalry was out of the way.

The lines of battle were being formed about 4 a. m., behind a line of skirmishers, and by daylight the Federal Army was in motion toward its allotted posts. The active aggression of the Federals in front of Hood's right for several days had led him to think that this might be the probable point of attack, and the feint of Steedman early in the morning of the battle led him further in his belief. The entire Federal Army was fairly in position by 8 a. m., and already there had been several clashes. It was not until about noon, however, that the fog lifted and disclosed the full intentions of Thomas. Before this, Hood had been weakening his center and left to sustain the attack on his right, but he now saw his mistake and endeavored to correct it. Already his front was hard pressed, and the strain on his left flank was almost up to the breaking point.

Montgomery Hill, on Hood's left, had been strongly fortified and

formed a salient against which Thomas's forces were vainly struggling. Attack after attack had been repulsed, but finally a crossfire enfilading the lines in two directions was secured, and this salient fell early in the afternoon. Montgomery Hill was Hood's headquarters and an important key-point in his line.

Hood, realizing too late that the main blow was being delivered on his left, endeavored to transfer troops back from his right flank to his left, causing a confusion which prevented the exercise of his greatest defensive powers.

After the feint of the morning on Hood's right, which was not pressed, certain advantages were secured in that vicinity during the forenoon. The Raines house, on Hood's right, was captured, and his lines in the vicinity were driven back. The fall of Montgomery Hill, however, dislodged his lines and he was forced to give ground, stubbornly disputing every step. Hood's fighting had been mainly on the offensive before this battle, but now he found that a defensive action did not give him sufficient scope for his peculiar ability. During the day one position after another was lost, until he had been pressed back about 2 miles in all in the first day's fight. His troops were badly demoralized and broken in organization, but they were saved from more serious disaster by the darkness of night on the 15th.

On the morning of the 16th, Hood had formed his lines about 2 miles to the south of his location on the day before; resting on two hills, Shy's on the left and Overton's on the right. He had withdrawn both flanks to prevent as far as possible their being turned, but his position was now so nearly surrounded that but little hope of victory remained.

The position of Hood's forces on the 16th was as follows: Stephen D. Lee, on his right, was holding Overton Hill on the Franklin pike; Stewart held the space between Granny White pike and Franklin pike; and Cheatham, on the left, was holding Shy Hill, protected by Chalmers on his left. Thomas's army from left to right consisted of Steedman, Wood, Smith, and Schofield, with Wilson's cavalry still further to the right and encircling Hood's left flank.

Early on the morning of the 16th Thomas brought his troops into close contact all along the line. His plans were to enfold both of Hood's flanks rather than to attempt to break his center. If one flank attack should be successful, Hood's retreat would be difficult, as there would then be a possibility of cutting him off

entirely. Although Hood's flanks were stronger than his center, still his retreat, which seemed imminent to Thomas, would be much harder to direct if Thomas could sufficiently extend his flanks to cut off his escape.

On Shy Hill, where Hood had formed his salient and rested his left, Schofield and Smith set up a cross-fire enfilading the Confederate lines in two directions. An attack on Hood's right was vigorously repulsed, although the attack had its effect in the final outcome. In order to strengthen his flanks Hood withdrew troops from Cheatham at Shy Hill until his strength there was so depleted that the Federals finally captured it with a bayonet charge, which together with a sharp attack upon the Confederate extreme left, turned the tide of battle. Hood's line was thus shattered, and his army forced back in utter confusion along the Franklin pike.

It is stated that Hood's retreat was nearly a rout. Arms and accoutrements were thrown away, the wounded abandoned, the dead left where they fell, and all energies concentrated on escape. His army was beaten and crushed. Although Hood kept up his rear-guard fight through the Overton Hills, the pursuit was maintained until the Tennessee River was reached. Rear-guard actions were fought at Hollow Tree Gap and at Franklin, but Hood destroyed his bridges behind him and the swollen rivers were effective rear guards.

This defeat put an effectual end to Hood's army for the remainder of the war, its power of offense having been completely destroyed. The loss on the Federal side, killed and wounded, was about 3,057, of whom less than 400 were killed. The loss on the Confederate side in killed and wounded was also comparatively small, but Thomas reported the capture of 13,000 prisoners and 32 cannon. It is said that not more than one-half of the impetuous force that was to raise the hopes of the Confederacy succeeded in crossing the Tennessee River.

Nashville is said to have been one of the most decisive victories gained by either side during the Civil War, and one of the most brilliant. It ended the hopes of the Confederacy in the West, and hastened the final surrender at Appommattox.

In conclusion it may be well to point out some of the lessons of this battle, so replete with warnings and fine examples. The first point of comment is that the most decisive victories are never won without hard fighting, and that vigorous attacks under well-designed plans are less deadly to the attacking force than half-

FROM THE "BIVOUAC" FOR AUGUST, 1885.

hearted or ill-directed efforts. This engagement has called forth high praise for the careful and well-laid plans of the Federal generals, the loyal and effective cooperation of all the subordinate commanders, and the high spirit and fighting quality of the Union forces. The battle moved off without special mishap, from the first feint to the final assault, in accordance with the principles of high generalship. It is a brilliant example of simple but effective strategy and well executed tactics.

The second point of comment is that Hood's delay at Tuscumbia before crossing the Tennessee River lost for him the campaign, by permitting his enemy to concentrate; and his failure at Spring Hill to cut off and crush Schofield's inferior force, together with his later repulse at Franklin, destroyed all probability of his accomplishing his ulterior designs. The delay was not necessary, and the Spring Hill mistake was the greatest error of his entire campaign. To permit an inferior retreating force, encumbered with hundreds of wagons, to escape in full view of a much larger force lying along its flank, when almost any attack would have thrown the retreating army into confusion, seems to have been a grave military blunder and one that cost Hood whatever advantage he had. This made his next move to Nashville a grievous mistake. He says that he hoped to beat Thomas in the battle which he knew would follow his advance on Nashville and in the meantime he was waiting for the promised reinforcements from Texas, which, it so happened, never were able to cross the Mississippi River. After Franklin his only reasonable hope was to leave Nashville in his rear, and by a dash threaten or cut Thomas's communications and force the Federal troops to fight on disadvantageous terms farther to the west. He had a desperate battle to fight in any case, and by drawing Thomas from his fortifications he had a much better chance for the advantage than by waiting for an attack.

Another point of comment is Thomas's reluctance to go to Schofield's assistance even before the battle of Franklin. It would seem that it would have been best to strengthen the army at the Tennessee River, and thus obviate any such critical situation as arose when Hood crossed the river on his march northward. As soon as Hood's advance was determined on, his movements should have been opposed by a much stronger force.

These observations, however, are open to the criticism that some other contingency might have arisen to change all the plans; but

in reviewing the events as they actually occurred, these conclusions present themselves as natural and logical from the military point of view.

There is also some question as to whether the military situation would not have been better met in the first instance by detaching more troops from Sherman's Army for the assistance of General Thomas. The force in front of Sherman after Hood's departure was of small account and offered no special opposition. The condition, of course, was known, and Sherman's march could have taken place just as successfully with 10,000 or 15,000 less troops, which would have added enormously to Thomas's fighting strength. It may be maintained that Sherman needed all of his force to aid Grant in an attack on Lee; but this is met by the argument that unless Thomas could beat his adversary Sherman's entire movement might fail. The fortunes of war often rest on slight accidents, and narrow escapes frequently occur; but it is folly to rely for success on the favorable outcome of such chances.

Another difficult point to understand at this time is why Hood did not make good his retreat after the first day's battle. Had he commenced his retreat that night he could have been far enough away to make his escape a far simpler matter; in which event his army would have remained a factor to be taken into consideration in future movements. It must be assumed that Hood relied on taking advantage of some accident or mistake that would enable him to break through Thomas's lines; but the result of the first day's action should have convinced him that he had nothing to gain by a longer contest.

Col. John James Abert.

BY

Lieut. WM. H. HOLCOMBE,
Corps of Engineers, U. S. A.

Col. John James Abert was born at Frederick City, Md., September 17, 1788. At the age of 19 he entered the Military Academy at West Point, only six years after the establishment of that institution by law. He remained at the Academy until his graduation, April 1, 1811, when he resigned from the Army and entered the War Office in a civil capacity. While thus engaged he became counselor-at-law in both the District of Columbia and the State of Ohio.

In 1814 he volunteered as a private soldier in the District of Columbia Militia for the defense of the Capital against the British. On August 24, 1814, he was engaged in the Battle of Bladensburg, Md., and his services on that occasion were acknowledged by the bestowal of a land warrant under the existing laws. On November 22d of the same year he returned to the Army, being appointed Topographical Engineer with the rank of Major. After the close of the war with Great Britain, the Topographical Engineers were employed in surveys of the Atlantic Coast and of the inland frontiers of the United States; Major Abert serving on a number of important surveys during that period of our nation's development.

The wide scope of his work in this connection is indicated by the following services, which form only part of those which he rendered during the period mentioned. He was an assistant in the geodetic survey of the Atlantic Coast from 1816 to 1818, and in making a reconnaissance of the East River, N. Y., in the latter year. He served as superintending Topographical Engineer of surveys in Chesapeake Bay in 1818, and of Dutch Island and the western entrance to Narragansett Bay, R. I., in 1819. From 1819 to 1821, he was superintending engineer of surveys of the East River, N. Y., Fall River, Mass., the Louisville Canal, Ky., Mount

Hope Bay, Narragansett Roads and Coxs Head, while in 1824-1825 his work was on the Chesapeake and Ohio Canal and the Patuxent River, and in 1826-1827 in the State of Maine.

In 1824, Major Abert was brevetted Lieutenant Colonel for ten years' faithful service in one grade, and in 1829 he was appointed to the charge of the Topographical Bureau. In 1831, due to the rapid increase in magnitude and importance of the duties of the Topographical Bureau, Colonel Abert was instrumental in having it separated from the Engineer Department and made a distinct branch of the War Department. Finally, in 1838, Congress organized a Corps of Topographical Engineers, with the officer, to whose care and management it mainly owed its existence, for its colonel and chief. Much interest and value is attached to the operations and work of this Corps under Colonel Abert's management and direction. The geographical and other information concerning this continent, which its officers collected and published, has been admired by the scientific world, and the practical benefits of their labors have been felt in many States of the Union.

Colonel Abert remained at the head of his Corps for upwards of thirty-two years, from 1829 until he was honorably retired from active duty on the 11th of September, 1861, after having become incapacitated by long and faithful service from further attendance at his office. As a citizen and a man, he was remarkable for the steadfastness of his friendships, for his candor and unostentatious hospitality, and for the simple piety which supported his declining years. It was with great regret indeed that the wide circle of Colonel Abert's friends, both personal and official, learned of his death at the city of Washington on January 27, 1863.

Excess Head in the Operation of Large Locks through the Momentum of the Column of Water in the Culverts.

BY

Maj. GEORGE B. PILLSBURY,
Corps of Engineers.

Certain unusual features observed in the operation of the locks of the Panama Canal were related by Colonel Hodges in the January-February number of the MEMOIRS, and are contained in the Annual Report of the Panama Canal. Prominent among them are the facts that the time of filling does not exactly follow the usual formula

$$t = \frac{2A\ (\sqrt{H} - \sqrt{h})}{ca\sqrt{2g}}$$

and that, at the end of the filling operation the water tends to rise above the level of the forebay, while at the end of the emptying operation it tends to fall below the level of the water in the tailbay. Investigation shows that the features can be fully accounted for by the kinetic energy stored and released by the column of water in the culverts. It will be shown that the effect of the momentum of this column of water is approximately equivalent to a constant excess head operating during the whole period after the flow has become established.

DERIVATION OF FORMULA.

Let h be the difference in level between the forebay and the lock at the time t;
$-dh$ the change in the time dt;
dQ the quantity of water transferred in the time dt;
w the weight of 1 cubic foot of water;
A the area of the lock;
a_1, a_2, etc., the cross sectional areas of the various portions of the culvert and branches;

r_1, r_2, etc., their hydraulic radii.
s_1, s_2, etc., the lengths of these sections.
v_1, v_2, etc., the velocities in these sections at the time t;
k_1, k_2, etc., the portion of the total flow that passes through these sections.

It is apparent that

$$dQ = -Adh = a_1 v_1 dt/k_1 = a_2 v_2 dt/k_2, \text{ etc.}$$

The flow from the forebay to the locks is subject to loss of head at entrance and exit, at contractions, valves, bends, etc., and through friction.

The loss of head at any section, a, where contraction or enlargement occurs, is, m being the appropriate coefficient,

$$mv^2/2g$$

The resulting resistance is

$$mav^2w/2g.$$

And the work required during the time dt to overcome this resistance is

$$mav^3wdt/2g.$$

Similarly, the work required to overcome the frictional resistance in a section of length, s, is (f being the appropriate frictional coefficient)

$$(fs/2r)\ av^3wdt/2g.$$

For the whole system this work is

$$(\Sigma mav^3 + \Sigma fsav^3/2r)\ wdt/2g = (\Sigma mk^3/a^2 + \Sigma fsk^3/2ra^2)\ \frac{w}{2g}\left(\frac{dQ}{dt}\right)^3 dt.$$

Placing

$$M = \Sigma mk^3/a^2 + \Sigma fsk^3/2ra^2$$

the total work done in overcoming resistance in the time dt, is

$$M\ \frac{w}{2g}\left(\frac{dQ}{dt}\right)^3 dt.$$

The external work done in the time dt is the transference of the quantity of water dQ through the height h, and is

$$whdQ.$$

The kinetic energy of the water in the culvert at the time t is

$$\Sigma wasv^2/2g = \frac{w}{2g}\left(\frac{dQ}{dt}\right)^2 \Sigma k^2 s/a = N\ \frac{w}{2g}\left(\frac{dQ}{dt}\right)^2$$

where

$$N = \Sigma k^2 s/a.$$

The change in this energy in the time dt is

$$N\,\frac{w}{g}\,\frac{dQ}{dt}\,\frac{d^2Q}{dt^2}\,dt.$$

The general equation for the work done in the time dt is therefore

$$whdQ = M\ \frac{w}{2g}\left(\frac{dQ}{dt}\right)^3 dt + N\,\frac{w}{g}\,\frac{dQ}{dt}\,\frac{d^2Q}{dt^2}\,dt.$$

Whence,

$$h = \frac{M}{2g}\left(\frac{dQ}{dt}\right)^2 + \frac{N}{g}\frac{d^2Q}{dt^2}$$

or,

$$h = \frac{A^2M}{2g}\left(\frac{dh}{dt}\right)^2 - \frac{AN}{g}\frac{d^2h}{dt^2}. \qquad (1)$$

The first integration of this equation gives

$$\left(\frac{dh}{dt}\right)^2 = \frac{2g}{A^2M}\left(h + \frac{N}{AM}\right) + ce^{\frac{AM}{N}h}.$$

Where c is the constant of integration. Or, placing, for convenience,

$$\frac{N}{AM} = o$$

$$\left(\frac{dh}{dt}\right)^2 = \frac{2g}{A^2M}(h+o) + ce^{\frac{h}{o}}$$

At the beginning of the operation of filling the water in the lock is at rest, and h is the initial head H. The consequent conditions

$$h = H \qquad \frac{dh}{dt} = O$$

Give for c the value

$$-\frac{2g}{A^2M}(H+o)\,e^{-\frac{H}{o}}$$

So that the equation becomes

$$\left(\frac{dh}{dt}\right)^2 = \frac{2g}{A^2 M}\left[h+o-(H+o)\,e^{\frac{h-H}{o}}\right] \qquad (2)$$

DISCUSSION.

I doubt whether the relation between h and t in terms of usual functions can be obtained from this equation. It, itself, indicates, however, the history of the flow. Consider a representative section of the main culvert, of area a. Under conditions of steady flow the velocity in this section would be

$$v = c\sqrt{2gh}$$

or,

$$v^2 = c^2 \cdot 2gh \qquad (3)$$

where c is the proper coefficient of flow. From this equation the usual formula for the time of filling is readily derived.

Recollecting that

$$v = \frac{A}{a}\frac{dh}{dt}$$

equation (2) may be written

$$v^2 = \frac{1}{a^2 M} \cdot 2g\left[h+o-(H+o)\,e^{\frac{h-H}{o}}\right] \qquad (4)$$

From the nature of the coefficient M, it is apparent that c^2, in equation (3) and $1/a^2M$ in equation (4) are identical. The two equations, for steady flow and for the actually varying flow, differ then in that the effective head of the latter is the actual head of the former increased by the constant quantity o, and decreased by the varying quantity $(H+o)\,e^{\frac{h-H}{o}}$. For a brief period at the beginning of the operation, this last quantity is a large fraction of the actual head, the effective head is less than the actual head and the flow consequently less than that due to the actual head. During this period kinetic energy is being stored in the culverts. As the flow continues and h diminishes, the value of this varying quantity decreases with great rapidity, and soon becomes negligible. The effective head is then the actual head increased by the constant quantity o, which may be designated the momentum head. The flow is greater than that due to the actual head, as it should be, since the stored energy in the culverts is being realized. As the period of storage of energy is very brief, while the release continues throughout the remainder of the operation of filling, the net time of filling is reduced. At the moment of equalization

of level, there is a remaining effective head, o, on account of which the flow is continued, and the water in the lock raised above the level of the forebay. If no water escapes from the lock, the rise should equal this excess head.

If the time of filling be reckoned from that at which the flow is well started, the value of $(H+o)\,e^{\frac{h-H}{o}}$ in equations (2) and (4) may be neglected, and the time of filling is then easily determined to be

$$t=\frac{2A\sqrt{M}}{\sqrt{2g}}(\sqrt{H+o}-\sqrt{h+o}) \qquad (5)$$

Which may equally well be written

$$t=\frac{2A\ (\sqrt{H+o}-\sqrt{h+o})}{ca\sqrt{2g}} \qquad (6)$$

Where a is the area of a culvert which, with a coefficient of discharge c, would be equivalent to the given system.

The value of o is, it will be recollected,

$$o=\frac{N}{AM}=\frac{\Sigma k^2 s/a}{A\ (\Sigma k^3 m/a^2+\Sigma k^3 fs/2ra^2)}$$

The calculation of the numerator of this expression is not difficult; the parenthesis, M, in the denominator can be calculated in detail, or arrived at from general considerations from relation

$$M=1/c^2a^2$$

where as above c and a are the coefficient and area used in the formula in its common form.

For the lock emptying, the head is similarly increased after a brief period by a constant quantity o, dependent upon the dimensions of the emptying culvert system, and the formula for the time of the operation is identical with that above deduced.

For locks in flight, A becomes $\frac{AA'}{A+A'}$ where A and A′ are the respective areas of the locks.

APPLICATION TO PANAMA LOCK.

It is of interest to apply these results to the observed behavior of the Pedro Miguel Lock as described and shown in the diagrams in Colonel Hodges' article. The value of the coefficient N for the side filling culverts is computed in the subjoined tabulation, the

flow being assumed equally distributed among the several branches.

Pedro Miguel Lock—Side Culvert.

Section.	k	a	s	k^2s/a	Sub-totals.
Intakes	1/3	144	35	.026	
Total for 3 intakes					.078
Culvert:					
Intake junction to valves	1	333	285	.860	
Valves to 1st lateral	1	255	220	.864	
1 to 2	10/11	255	65	.211	
2 to 3	9/11	255	65	.170	
3 to 4	8/11	255	65	.135	
4 to 5	7/11	255	170	.270	
5 to 6	6/11	255	72	.081	
6 to 7	5/11	255	72	.058	
7 to 8	4/11	255	72	.037	
8 to 9	3/11	255	72	.021	
9 to 10	2/11	255	72	.009	
10 to 11	1/11	255	72	.002	2.718
Laterals:					
Entrance to 1st outlet	1/11	41	46	.0093	
1 to 2	4/55	41	20	.0026	
2 to 3	3/55	41	20	.0015	
3 to 4	2/55	41	20	.0006	
4 to 5	1/55	41	20	.0002	
Total for 11 laterals				.0142	0.154
				N=	2.950

The coefficient M is obviously not dependent upon the section of least area only, but upon the characteristics of the whole system. The coefficients, m and f, which enter into it, are somewhat uncertain. It is easy to select values for these coefficients, within the range that theory and practice have designated, which will give a value of M consistent with the observed results. It does not appear, however, that any very useful purpose is served by going into such details, and the value of M will be obtained from observed results.

Referring to the diagram, Fig. 4, on page 7 of Colonel Hodges' article, showing the observed time of filling the lock from the side culvert, with both valves open, it will be seen that the second set of gates opened when the water in the lock stood at about 84.3. The level in the forebay must therefore have stood slightly below this

level. During the first part of the filling operation it stood at 84.0. It would appear therefore appropriate to take the mean level of the forebay, during the operation, at 84.1, and the time of equalization ($h=0$) at 11.8 minutes after the time when the flow was established, at which latter time the level of the water in the lock was 51.7. The value of H then becomes 32.4, and of t, 708 seconds. As A is 123,000 square feet, the value of M can be deduced from equation (5) and is .00002344. The value of the momentum head, o, is then 1.05 feet. The observed excess head at the end of the flow appears to have been in the vicinity of .6 feet, but a considerable amount of water must have been liberated through the opening of the gates.

For the time of filling between 51.7 and 70, we have

$$\begin{aligned} H&=32.4 \\ h&=14.1 \\ o&=1.05 \end{aligned}$$

whence $t=4.67$ minutes, as compared with 4.6 minutes recorded.

For the time of filling between 70 and 84.1, we have

$$\begin{aligned} H&=14.1 \\ h&=\ 0 \\ o&=\ 1.05 \end{aligned}$$

whence $t=7.08$ minutes, as compared with 6.9 minutes recorded.

For the time between equalization and the exhaustion of the excess head, we have

$$\begin{aligned} H&=0 \\ h&=-o=-1.05 \end{aligned}$$

whence $t=2.5$ minutes, which again appears to agree with the diagram.

It appears, therefore, that the formula presented agrees quite closely with the observed results.

For a small lock it will be found that o has a negligible value, and the effects noticed in the Panama locks are not, consequently, realized.

An extension of the discussion would, I think, account for the differences in level between the water in the locks and the water enclosed between the upper and lower sets of gates, respectively.

SAN JUAN HARBO
PORTO-RICO.
Scale.
1000 0 1000 2000 3000 4000 Ft.
1 Mile
CABRAS I.
Morro Pt.
Light House
SAN JUAN ISLAND
FORT
CAÑUELO
PALO SECO Pt.
La Puntilla
Bregg to 30 ft.
Punta Largo Shoal
Reef
CANO DE SAN ANTONIO
ISLA GRANDE
COMADO BAY
MIRAFLORES BAY
MIRAFLORES
SAN JUAN HARBOR
SANTURCE
H.W.L.
M.L.W.
CATANO
The 6 ft. contour is shown thus:
" 12 " " " " "
" 18 " " " " "
" 24 " " " " "
" 30 " " " " "

FIG. 1.

Pipe Line Dredges.

BY

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Corps of Engineers, U. S. Army;
Assoc. M. Am. Soc. C. E.

PART I.*

I. DESCRIPTION OF A DREDGING PROJECT FOR SAN JUAN HARBOR, PORTO RICO.

As will be seen from the accompanying map (Fig. 1), San Juan Harbor is a land-locked bay of considerable extent. The deep water area is, however, very limited. The entrance channel has a depth of 28 feet at mean low water, but, except for the channel to the docks and a very limited anchorage south of the point "La Puntilla," there is no 30-foot area inside of the harbor. Opposite La Puntilla the deep water areas divide. A channel leads to the docks and, in prolongation of the entrance channel, depths of 18 feet or over extend for a considerable distance to the southeast and east. Between these two channels Punta Larga Shoal extends into the bay from the east. If the harbor is to be extensively improved, the removal of Punta Larga Shoal is the first requisite.

Surrounding the harbor are marshes covered with mangrove bushes, and known locally as Manglares. While the mean range of tides is only about 1.1 feet, these swamps may for all practical purposes be said to be covered at high tide and dry at low tide. Under the Spanish law, all lands below the high water line belonged to the Crown of Spain and, if reclaimed by natural or artificial process, they remained crown lands. The Manglares are now publicly owned, and it is obvious that here was a fine opportunity for dredging and reclamation to be combined.

Surveys and borings for a new project were completed before the

* Part II, relating to characteristics of centrifugal pumps, will appear in an early number.—ED.

writer arrived. To dredge the entire area surveyed, including Punta Larga Shoal, to 30 feet would require the removal of about 5,531,000 cubic yards of material. The borings showed it to consist of mud and clays of varying degrees of compactness. It was finally separated into two classes—"soft" and "tough." It was assumed that an hydraulic dredge working in the tough material could excavate only 60 per cent as much material as when working in the soft material.

The areas to be excavated and those to be reclaimed were subdivided into sections and a tentative distribution of spoil was made. The distances to which the spoil must be pumped were roughly estimated and the following table resulted:

Cubic yards.		Length of discharge pipe.
Soft.	Tough.	
311,000	607,000	2,000
81,000	157,000	4,000
226,000	444,000	5,000
107,000	368,000	6,000
56,000	194,000	7,000
148,000	506,000	8,000
65,000	225,000	10,000
274,000	941,000	13,000
622,000	199,000	15,000
1,890,000	3,641,000	

It was desirable that the spoil be disposed of as indicated, but not imperative. The distances it must be pumped could be reduced by increasing the height of the fills or by wasting material by pumping it across San Juan Island to the ocean. The cost of pumping it various distances was a matter of vital importance. A search through current engineering literature revealed little or nothing on the subject and, in preparing a project, the writer was forced to depend largely upon theory.

After the report on the project had been submitted the writer continued in an intermittent manner to look for additional light on the subject and this paper is the result. It is by no means a complete discussion of the subject of pipe-line dredges. It merely treats of certain features of dredge design that have been studied by the writer. Many of the subjects treated may be "open books"

to some of the readers. The reason for including them is that the writer has personally found difficulty in understanding them or has found that others do not agree with his conclusions.

If this paper be instrumental in bringing about a better underderstanding of the theoretical side of dredge design it will have accomplished its purpose.

II. QUESTIONS RAISED BY THE SAN JUAN PROJECT.

The primary effect of any increase in the length of pipe-line is an increase in the friction loss on the discharge end. This results in an increase in the total head against which the pump must operate. This raises the following questions:

(a) What will be the increase in head due to a lengthening of the discharge pipe?
(b) What will be the effect upon the efficiency of the pump of a stated increase in head?
(c) How can a pump be run at maximum efficiency with varying head?
(d) How much water can a given pump deliver through various lengths of discharge pipe of given diameter?

The above questions could be answered with a fair degree of accuracy if we were dealing with a pumping plant delivering water only. We are, however, dealing with a dredge which must deliver mixtures of earth and water and the following additional questions must be considered:

(e) What is the minimum pipe-line velocity required for the transportation of the classes of material which we propose to dredge?
(f) What is the maximum percentage of solids that can be transported with a given velocity?
(g) What changes in friction losses must we expect due to the presence of solids in the pipe line?

There are a thousand and one other questions that must be considered in any dredging contract. While they are not ignored by the writer, no attempt will be made to answer them here. Throughout this paper the writer assumes that the cutter is capable of loosening all the material the pipe-line can carry.

III. THE VALUE OF THERMAL, MECHANICAL AND HYDRAULIC EFFICIENCY.

Conclusions.

1. It is of the utmost importance that a dredge deliver at all times the maximum possible yardage.

2. As a means by which fuel bills may be reduced, thermal, mechanical and hydraulic efficiency are of small value.

3. As a means by which the yardage may be increased, thermal, mechanical and hydraulic efficiency may be of great importance.

The gross earnings of a dredge are dependent entirely upon the number of cubic yards excavated. The gross expense of operation is made up of many factors. Failure on the part of the operator to handle the cutter properly might easily cut the gross earnings in half, while gross expenses remained unchanged. A reduction in the combined efficiency of the boiler, engine, and pump may result in a reduced output of volume of mixture or it may follow as a result of an effort to increase the output at the expense of the combined efficiency. As will be seen below, a small increase in output will justify a large increase in coal bills. High thermal efficiency is a means by which a large output from a given dredge may be obtained. The end to be attained is the maximum output.

If the current in the pipe-line is not already fully loaded the operator may be able to overcome a loss in volume of mixture by increasing the percentage of solids. If the current is already fully loaded, a reduction in gross earnings will follow any decrease in volume of mixture. If the operator is able to keep the current fully loaded any increase in volume of mixture pumped will result in an immediate increase in gross earnings. The following figures have been compiled to determine the relative value of thermal efficiency and total output.

The operating cost of eight hydraulic dredges in use on the Upper Mississippi during the fiscal year 1913, is given in *Engineering and Contracting*, September 24, 1913. This cost may be distributed as follows:

Fuel	$23,411.23	
Engine room supplies	4,362.16	
		$27,773.39
Other expenses		113,097.76
Total		$140,871.15

	Per cent.
Fuel and engine room supplies	19.6
Other expenses	80.4
Total	100.0

The operating cost of the dredge *Pascagoula* for the fiscal years 1910 and 1911 is given in PROFESSIONAL MEMOIRS, No. 14, March-

April, 1912. Distributed in the same manner and adding 10 per cent of the first cost for depreciation and interest, the results are:

Pascagoula, 1910.

Fuel	$11,320.17	
Engine room supplies	3,702.46	
		$15,022.63
Other expenses		69,956.40
Total		$84,979.03

	Per cent.
Fuel and engine room supplies	17.7
Other expenses	82.3

Pascagoula, 1911.

Fuel	$14,789.73	
Engine room supplies	3,786.36	
		$18,576.09
Other expenses		97,809.78
Total		$116,385.87

	Per cent.
Fuel and engine room supplies	15.9
Other expenses	84.1

An increase in engine speed may be made at a sacrifice in pump efficiency. It may also result in a decrease in engine efficiency. The losses will, however, be relatively small and an increased output will always result. The above figures show that an increase of 1 per cent in output will justify an increase of 5 per cent in coal bills, etc. They substantiate the maxim of the dredge captain, that his boilers, engine and pump must at all times be pushed to their maximum limit. They also justify the designer when he sacrifices thermal, mechanical and hydraulic efficiency in order to obtain a machine that will operate the maximum number of hours per month.

On the other hand, it is the size and first cost of boilers and engines that frequently fixes the limit of the power available on new dredges. On dredges already constructed, one of the principal means now open for increasing the output is by increasing the efficiency. The subject of efficiency can not, therefore, be lightly set aside.

The writer has in mind an instance where the combined efficiency of engine and pump is less than 45 per cent. In this instance an efficiency of 55 per cent or over should be readily attainable by a change of pump design. Calling the present capacity of that plant 100, the capacity could be increased to 122 by the expenditure of a relatively small sum. The importance of efficiency in this case is obvious.

IV. WATER HORSEPOWER—HOW DETERMINED.

In the log of a dredge a record is frequently kept of the suction head or vacuum and the discharge head or "back pressure." Whether this record is kept or not, the gauges are always in front of the operator to assist him in intelligently handling the machine. In order to avoid possible misunderstandings it will be well to discuss briefly the relationship between these gauge readings, the materials, fluid and solid, discharged by the dredge and the power developed.

Take the discharge pipe and the back pressure gauge first. The gauge is usually located near the discharge flange of the pump and reads to pounds per square inch, or to feet of water, or both. For computing the flow of water in pipes, the latter calibration is the one most often used. So far as the discharge pipe is concerned, the work done by a centrifugal dredging pump is equal to the work that would be performed by a piston having the same cross-sectional area as the pipe, moving in the direction of flow with a velocity equal to the mean velocity in the pipe, and working against the pressure recorded by the gauge. If the pressure is recorded in pounds per square inch, the area of the piston will ordinarily be figured in square inches. If the pressure is recorded in feet of water the area of the piston will ordinarily be figured in square feet. In one case the work performed is represented by:

Work in ft. lbs. per sec.=(Pounds pressure×area in sq. in.)× ft. traveled per sec.

In the other case—

Work in ft. lbs. per sec.=(Ft. head×62.5×area in sq. ft.)×ft. traveled per sec.

The weight per cubic foot of the fluid pumped does not enter into either equation.

We may, however, wish to subdivide this work into two parts; the work required to overcome the frictional resistances of the discharge pipe, and work required to lift the fluid a certain number of feet above the level of the pump. In such cases the specific gravity of the fluid must be taken into account.

The work required to overcome the static head=the static head× 62.5×the specific gravity of the fluid×the number of cubic feet pumped.

The difference between this and the total amount of work done is equal to the work expended in overcoming friction.

The suction or vacuum gauge measures the velocity and entry heads and all losses in the suction pipe. If we should desire to analyze these losses, it is necessary to take the specific gravity of the fluid into consideration. The work expended in creating velocity is the velocity head times the weight of the fluid actually pumped. Materials heavier than water may be lifted from the end of the suction pipe to the surface of the water; from the surface of the water to the pump an actual static head must be overcome and the fluid may be heavier than water.

In case the suction and discharge pipes are of the same diameter, the total work done by the pump may be obtained by taking the arithmetical sum of the vacuum and pressure gauge readings and applying the formulæ for the discharge pipe as given above. If the discharge pipe is of a different diameter than the suction pipe, the pump may have been required to produce a greater or a less velocity than that for which it has already received credit through the intermediary of the vacuum gauge. In correcting for this change in velocity head the specific gravity of the fluid must be considered. This correction will ordinarily be relatively small.

Throughout this paper it will be assumed that the suction and discharge pipes are of the same diameter and the following symbols will be used:

W. H. P.=Water horse power.

Q=Cubic feet of water or mixture discharged per second.

H=total head against which the pump is working.

$$\text{Then: W. H. P.} = \frac{Q \times H \times 62.5}{550}$$

This equation is independent of the specific gravity of the mixture or fluid pumped.

V. ENGINES—CONSTANT HORSEPOWER *v.* FIXED CUT-OFF.

Conclusions.

The boiler horsepower should be sufficient for all reasonable demands. The engine should be of sufficient size to utilize at all times the full boiler horsepower. The engine should be adjustable in speed. The use of a fixed cut-off at all speeds is uneconomical. An increase in pump speed will always result in an increase in output, if the other conditions remain unchanged. The permissible rotative speed of any pump and engine is limited, but every effort should be

made to approach the maximum speed as nearly as possible with the power available.

The use to which the dredge will be put, the first cost, the space available, and the difficulties encountered in handling large plants in certain places, set a limit to the size of boilers and engines installed on any dredge. It is easier to increase the size of engines than to increase the size of boilers. After a boiler has been installed its power is a fixed quantity. After deducting a variable quantity for use in running the auxiliaries, all the remaining boiler power becomes available for use on the main pumping engine.

The main pumping engine should be able to use all the power the boilers can furnish whether the auxiliaries are running heavy or light, whether the pipe line be long or short and whether the fuel be good or bad. The only exception is when the cutter is unable to fully charge the stream of water flowing through the pipe line; any increase in pump output under those circumstances is, of course, useless.

The characteristics of a centrifugal pump, within the working range of dredging operations, are such, that, as the pipe line is shortened (the head lessened), the power required to operate the pump at constant speed increases. If the additional power be not supplied the pump will slow down. If the engine driving this pump be operated at constant cut-off the slackening of the speed reduces the power and a further slowing down results. The output of the pump may be reduced, due to this condition, to a point considerably below what it would be if the full power were utilized.

One possible advantage of the constant cut-off is that if, due to incipient clogging of the discharge pipe, the back pressure begins to rise, the pump and engine will automatically speed up and help to clear the pipe. From the writer's observation this feature is not of much value.

The writer favors a design calculated to use the full available power all of the time and in what follows it will be assumed that the brake horsepower (B. H. P.) available for driving the pump is a constant.

Conclusions—Sections VI, VII and VIII.

1. The friction losses due to the flow of water through dredge pipes may be obtained by use of the Williams-Hazen formula, using the value C=110.

2. For pipes 24 inches in diameter or less, a velocity of 10 feet per second is sufficient for all classes of material ordinarily encountered.

For pipes greater than 24 inches in diameter, a velocity of 12 feet per second should be used where coarse sand is encountered: This conclusion is tentative only. The above velocities are considered safe.

3. With the above velocities, or higher ones, the friction loss when pumping mixtures of sand and water may be obtained by adding 1 foot per thousand feet of pipe for each 1 per cent of sand in suspension to the friction loss due to the flow of water only at the same velocity.

4. Mud or clay may be transported at less velocities than the above and the friction losses will be less, approaching the losses for water only as a limit.

NOTE: For a diagram from which the friction losses due to the flow of water only or due to the flow of water carrying 10 per cent of sand, see Fig. 5.

VI. THEORY OF THE SUSPENSION OF SOLIDS.

The subject of how solid particles heavier than water are transported through the discharge pipe of a dredge will now be considered. Most of the writer's ideas have come from an article, "The Suspension of Solids in Flowing Water," by Elon Huntington Hooker, Ph. D., published in *Transactions of the American Society of Civil Engineers*, Vol. XXXVI. Mr. Hooker's article deals particularly with sediment, sand, etc., carried by rivers, and gives a very full list of experiments and papers on the subject. While the experiments deal principally with open channels and velocities much lower than those encountered in dredge pipes, many of the conclusions and results are applicable to the latter.

In nature, sands, etc., of spherical or other regular form are not encountered. In order to develop theories and in order to conduct experiments that are sufficiently consistent to enable laws to be established, it is ordinarily necessary to assume or actually manufacture particles of regular form. A law having been deduced, it is then possible to apply it to natural sands, etc., by the use of coefficients experimentally determined by using natural materials. In dredging, the conditions change so rapidly that there is no practical necessity for great refinement in theory or in the determination of coefficients applicable to sands, etc., that have been obtained from some particular, restricted locality.

It is well recognized that the velocities of the filaments of water flowing nearest to the sides of the restricting channel are retarded and that their velocities are less than those filaments at a greater

distance from the sides. In a pipe this is equivalent to saying that the maximum velocity is at the center of the pipe. The amount of retardation and the consequent relative velocities of adjacent filaments of water increases very rapidly as the side of the channel is closely approached. The vastly greater proportion of materials handled by dredges consist of individual particles less than .05 inch in diameter, and with filaments of water that distance from the sides of the channels the retardation of velocity must be very great. The writer knows of no experiments where such velocities have been successfully measured.

In 1848 Inspecteur General Dupuit published his "*Etudes Theoriques et Pratiques sur le Mouvement des Eaux,*" and in that publication he emphasized the importance of the effect of relative velocities of adjacent filaments of water upon the suspension of solids. The theory may be briefly described as follows:

Assume (Fig. 2) a solid particle heavier than water acted upon by

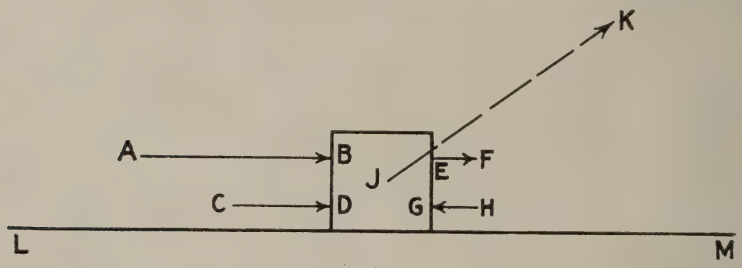

Fig. 2.

a current of water the filaments of which have different velocities. In the figure, AB may be considered as representing the resultant of forces acting upon the upper half of the upstream face of the particle and CD as the resultant of forces acting upon the lower half. Observations have shown that the actual velocity of a body in suspension is greater than the mean velocity of the current impinging upon it. On the downstream face the forces acting may be represented by HG the resultant pressure acting against the lower half, and EF a suction effect representing the resultant pull on the upper half. The particle under these conditions would follow a path JK. No attempt has been made to show actual quantitative results.

The above tendency is independent of the position of the particle with respect to the sides of the channel and merely indicates a tendency of a submerged body to move towards the filaments having the greatest velocity. If LM represents the side of a dredge pipe, the tendency of particles to leave the side and congregate in the center is apparent. As stated above, the relative velocities of different

filaments are much greater as the sides of the channel are closely approached and therefore the power of the water to hold solids in suspension will be greater near the bottom of the pipe than near its center.

There is in such a pipe another motion not easily described, and that is, a system of eddy currents. At each joint and at each irregularity of any sort the filaments are distorted and forced to suddenly change their direction of flow. Mr. Hooker describes the corresponding phenomena for open channels and by changing the phraseology so as to describe his "downward" forces, thrusts, etc., as forces, etc., acting from the center outward, and his "upward thrusts" as forces acting from the circumference of the pipe towards the center, his discussion becomes applicable to pipes. Mr. Hooker's discussion is as follows:

It is seen at once that vertical motion of the water itself can only be a local phenomenon. The resultant of all the external forces acting upon a given stretch of river will have a direction down stream, and will approach zero as the motion approaches uniformity. Its vertical component will be downward, not upward. The same thing will be true of all the complex motions in the interior of the liquid prism considered in the aggregate. It is true, however, that the reverse downward current need not equal in velocity the original upward one. The diminution in velocity may be counterbalanced by an increase in the moving area. This is an important point affecting the power of suspension. While there is no resultant vertical motion of the water, there is a resultant vertical thrust exercised upon sediment in the aggregate and acting as an efficient cause of suspension.

The thrust exercised by a current upon an immersed solid is

$$P = k\ F\ \gamma\ \frac{v^2}{2g}$$

and obviously varies with the square of the velocity. An eddy, having an upward component of velocity represented by 4, would exert an upward thrust, available for suspension, proportional to 16. In descending, the same water might cover twice the area, and have a downward velocity of 2. Then the corresponding downward thrust would be proportional to 4. The resultant of such a local system would be an upward thrust available for continuous suspension. Eddies are induced by the rugosities met with in the earth and air profiles. The abrupt changes in the bottom, and its relatively unyielding character, cause the upward currents to have the

greater velocity in general, though they must of necessity be restricted in relative extent.

The fact that the descending areas must be much greater than the ascending ones in order to give rise to this force shows that, in the major part of the stream, an active force is at work tending to carry material to the bottom, aside from the force of gravity acting upon the particles. This suggests, though it does not prove, the existence of another cause of suspension, and such a cause is actually found, as shown later.

Here, then, in obedience to the law which governs the thrust exercised by water impinging upon a solid, there is a resultant force acting upward in flowing streams, and capable of performing an efficient part in the continuous suspension of solid matter. While the aggregate of all the interior motions of the fluid can have no upward resultant, still the thrusts exercised upon solid suspended matter by these motions can and do have such a resultant as shown.

It is evident that this force would disappear in circular conduits, flowing full, since an uniform profile may be assumed. The eddies would be thrown off with like intensity at all points toward the center.

Instead of carrying the solid particle in suspension it may be rolled along the bottom. In dredge pipes this is particularly true of gravel and cobbles which are frequently encountered in small numbers and occasionally in pockets or beds. Experiments indicate that for this method of transportation the ability of a current to move the particle varies with the sixth power of the velocity, the weight of the particle being used as its measure of size. This would mean that the diameter of the particle may increase with the third power of the velocity. M. Sainjon decides that in a stream with gravel bottom, the gravel being 6.72 inches in diameter, scour will begin when the velocity of the current reaches 6.56 feet. As 6.56 feet per second would be a very low velocity for a dredge pipe and as the bottom of such a pipe would offer very much less resistance to rolling friction than would be offered by the gravel covered bottom of a stream, no difficulty from occasional cobbles should arise so long as the bottom of the pipe is kept reasonably clear.

If spherical grains be considered and the bottom of the pipe is allowed to become covered, the surface will no longer be a smooth one and the upper layer of particles would rest in the depressions between particles of the next lower layer. This is indicated in Fig. 3. Not only is the surface exposed to the action of the current reduced, but at the same time it becomes necessary to roll the particle up an incline as indicated by the arrow in order that

it shall once more be put in motion. The angle of inclination has been estimated as 37 degrees.

Mr. Hooker, on page 275 of his article, gives the following:

Experiments reported by Mr. G. F. Deacon, in connection with studies for the Manchester Ship Canal, give an accurate description of the detailed action of flowing water upon a bed of sand. His summary of results will be reproduced here:

The observations were made in a long flat-bottomed trough with glass sides by means of which the behavior of the sand could be accurately observed. The sand was from the estuary of the Mersey, the quantities moved were weighed and the surface velocities of the water carefully measured. When water flowed with a steadily increasing velocity over a surface of such sand, fine pieces of broken shell were first moved, and the surface velocity required to produce such movements was considerably less than 1 foot per second. At such velocities, however, the sand proper was perfectly stable, and however long the flow continued it remained undisturbed; but the

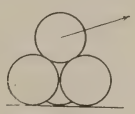

Fig. 3.

fine pieces of shells at the surface of the sand moved in spasmodic leaps, accumulating wherever the velocity was somewhat less.

The first movement of sand began at a surface velocity of 1.3 feet per second. This movement was confined to the smaller isolated grains; and if the same velocity was maintained, the grains so moved ranged themselves in parallel bands perpendicular to the direction of the current, each band taking the form of the well-known sand ripples of the sea shore or sand-bottomed stream, with its flat slope upwards and its steep slope downwards in the direction of the current. At this velocity the profile of each sand ripple had a very slow motion of translation, caused by particles running up the flatter slope and toppling over the crest. The steep downward slope was, therefore, being constantly advanced at the expense of the denudation of the less steep upward slope. At a surface velocity of 1.5 feet per second the sand ripples were very perfect and traveled with the stream at a speed of about $\frac{1}{2160}$ of the surface velocity. At a surface velocity of 1.75, the ratio was reduced to about $\frac{1}{1050}$, and at a surface velocity of 2 feet to $\frac{1}{480}$. A critical velocity was reached when the surface of the water moved at 2.125 feet per second, when the sand ripples became very irregular, indicating greatly increased unsteadiness of motion of the water. Up to this

point the whole amount of scour was represented by the volume of sand waves multiplied by an exceedingly low velocity, always less than the $\frac{1}{480}$ part of the surface velocity of the water. At about this critical velocity of 2.1 feet per second, the particles rolled by the water up to the flat slope, instead of toppling over the steep slope, were occasionally carried by the water direct to the next crest; and as the velocity of the water was gradually increased, an increasing bombardment of each crest by the crest behind it took place.

At about 2.5 feet per second, another critical velocity was reached and many of the little projectiles cleared the top of the first or even of the second crest ahead of that from which they were fired. At surface velocities of 2.6 to 2.8 feet per second, the sand ripples became more and more ghost-like, until, at 2.9 feet per second, they were wholly merged in particles of sand rushing along with the water in suspension. After this the scour was of a totally different character; the sand and water became mixed, and a constant process of lifting, carrying and depositing of individual particles ensued, the sand being stirred to a depth and lifted to a height dependent upon the velocity.

Sand waves have a progressive motion with the current and sand may be transported in this manner with very low velocities, but the quantity passing any given point in a given time is too small to make the process an economical one in dredging operations.

Quoting again from Mr. Hooker's article, we find the following:

A particle whose specific gravity is greater than one will sink in quiescent water when viscosity is overcome. It will, when falling freely from rest, after a very short period of acceleration, attain a state of velocity practically uniform. The same particle should be kept suspended indefinitely by a constant current, whose vertical component has a velocity equal to that asymptotically approached by the falling particle.

Experiments have been made to determine the velocities of currents necessary to keep particles of various sizes and specific gravities suspended when acting in a direction opposed to gravity.

The table . . . gives the results of experiments by Mr. Thoulet. It represents the maximum velocities in millimeters per second of . . . currents capable of holding suspended, at a fixed point in a tube, spherical grains . . . The velocities are taken from diagrams constructed from results of actual experiment.

Velocity of current (V) in MM. per second.

Radius of grains MM.	S. G. 1.5 V	S. G. 2.0 V	S. G. 2.5 V	S. G. 3.0 V	S. G. 3.5 V	S. G. 4.0 V
0.1	13	23	32	40	47	53
0.2	23	40	55	69	79	85
0.3	32	57	78	94	106	116
0.4	40	70	94	114	129	143
0.5	51	82	107	132	151	168
0.6	57	92	120	147	168	187
0.7	61	100	133	163	185	205
0.8	65	108	142	174	198	221
0.9	68	116	152	185	211	237
1.0	72	123	161	195	224	252
1.1	75	128	169	203	234	264
1.2	78	133	175	211	243	273
1.3	80	137	180	218	252	281
1.4	82	141	184	224	260	288
1.5	84	144	188	229	267	295
1.6	86	147	192	234	271	300
1.7	88	149	195	237	274	304
1.8	90	151	197	240	277	308
1.9	92	153	199	243	280	312
2.0	94	154	201	246	283	315
2.1	95	155	202	249	286	317
2.2	96	156	203	251	288	319
2.3	97	156	204	253	290	321
2.4	98	156	205	254	291	323
2.5	98	156	206	255	292	324

Sand grains are not spherical and the indications are that their velocities of fall through water are about 0.7 as great as the velocities indicated in the above table.

In a current of water moving in a strictly horizontal direction and with all filaments moving parallel and at the same velocity a solid particle heavier than water would settle to the bottom at the same rate as in quiescent water. Its absolute motion in a vertical direction may be obtained from the above table. Its absolute motion in a horizontal direction is that of the surrounding current. A grain of sand in such a current must sooner or later settle to the bottom after which it may or may not be rolled along according to circumstances.

We have now considered three forces acting on the particles in suspension in a circular pipe. Two of these forces tend to draw or push the particle from the circumference of the pipe towards the

center and the third is the ever present force of gravity tending to draw the particle to the bottom. Whenever there is no noticeable concentration of solids in any part of the pipe it would seem reasonable to expect that the eddy currents are of such proportion as to cause the filaments to cross and recross from side to side and from top to bottom keeping the entire cross section churned up, as it were, until the effects of the other forces are not readily observed. The excessive wear on the bottom of dredge pipes proves that in the normal case there is a considerable concentration of heavy particles, near the bottom of the pipe.

The above theories deal with the forces by means of which particles heavier than water may be held in suspension by the water. In general they indicate a tendency for the particles to settle to the bottom of the pipe and a repellant action, as it were, between the inner surface of the pipe and the particle. In general the progress of the particle is a succession of leaps, each time it approaches the inner surface of the pipe it is projected towards the center. On one very important point the theory is absolutely blank. How many particles can the current carry per cubic foot?

Taking sand for example, some idea is obtainable as to the relative strength of forces required to keep in suspension individual grains of varied sizes. For a vertical pipe and spherical grains, this data is given in exact figures. But nowhere is there an indication as to the space between particles required in order to bring these forces to bear upon the individuals. Taking a cubic foot of ordinary clean river sand the solids present represent about 0.6 cubic foot. Add 1.4 cubic feet of water and, while thoroughly agitated, a uniform mixture having a total volume of 2 cubic feet results. According to the usual phraseology based on place measurement of the sand, this would be called a 50 per cent mixture. Is there a sufficient proportion of water surrounding the individual grains to bring the above forces into action? Based on judgment only, the writer would say for a vertical pipe, yes; but for a dredge pipe, no. If to this cubic foot of sand there had been added 3.4 cubic feet of water, the percentage of sand would have been reduced to 25 and precedents for such percentages may be found in actual dredging data.

The following from a paper "Works for the Purification of the Water Supply of Washington, D. C.," by Messrs. Allen Hazen and E. D. Hardy, Members of the American Society of Civil Engineers, published in *Transactions American Society Civil Engineers*, Vol.

LVII, and the discussion of the paper throw some light on the above subject.

Page 342, Messrs. Hazen and Hardy:

By authority of Colonel Miller, tests were made to determine . . . the frictional resistances of mixtures of sand and water in pipes, and the minimum velocities at which such pipes could be regularly operated without stoppage by separated sand.

In making these tests an ejector was connected by hose to a length of wrought-iron pipe, which finally discharged into a tank on a wooden trestle. The outlet from this tank was through a circular hole in a disc of sheet-iron upon the bottom, which acted as a standard orifice. Pressure gauges connected with the pipe at intervals gave better results as to the loss of head than could be secured by differential gauges of glass tubes. There is a throbbing in pressure, particularly at moderately low velocities, perhaps due to a partial deposition and subsequent picking up of sand particles, and to separation into more or less dense mixtures, which made it difficult to get satisfactory readings in the glass tubes. The pressure gauges were less sensitive and therefore much more easily read, and on the whole, sufficiently accurate.

The time required to throw a measured volume of sand was observed by one man, while others read the gauges on the discharge pipe at 10-second intervals. At the same time the head over the standard orifices was noted, and from this the total volume of flow was computed.

Such experiments were made with 3-inch and 4-inch pipe, from which diagrams of friction were prepared for use in designing the piping for the filters. They are not as accurate as could be wished, and further data of this kind are to be desired.

With an average velocity as low as 2 feet per second, a pipe would promptly fill up with sand and shut itself off. This was true, whatever proportion of sand was present in the water. At 2½ feet per second flows of sand and water were sometimes maintained, but always with great fluctuations in friction, which, the writers think, indicates a partial deposition of coarse particles and an occasional picking up and carrying forward of an accumulation. Occasionally, at this velocity, the pipe would fill so as to shut off the flow.

At velocities of 3 feet per second the flow was much steadier, and stoppages almost ceased. At 4 feet per second and upward, mixtures of sand and water acted quite as a fluid, and the flow was almost as steady and regular as that of so much water, although the frictional resistance was much greater.

On page 391, Mr. J. A. Vogleson gives tables showing percentages of sand from 6.1 to 19.6 that were successfully transported through 403.1 feet of 4-inch pipe, laid horizontally at velocities of from 4.12 to 4.91 feet per second. This table will be referred to

later. On the same page in another table Mr. Vogleson shows the results obtained with 4-inch pipe having first a vertical lift of 5.71 feet, then a horizontal section 9 feet in length. With velocities less than 2 feet per second he succeeded in delivering more than 50 per cent of solids on several occasions. On page 292, Mr. Vogleson concludes that 10 cubic yards per hour can be delivered through 403.1 feet of 4-inch pipe by the use of approximately 1,000 gallons of water per cubic yard. This is equivalent to about 18 per cent of solids. His difficulty was apparently to get the sand into the pipe and no particular difficulty appears to have been encountered in transporting it.

The following results, obtained during the construction of the Gatun Dam, throw a little light upon what may be expected in actual dredging operations. The writer obtained his information by correspondence with Major George M. Hoffman, Corps of Engineers. All discharge pipes were 20 inch, and under the worst conditions one dredge and two relay pumps were operated at the same time on a pipe line 9,000 feet in length and having a vertical lift of 100 feet.

The velocity of flow in the pipe is described as:

> Normally 7 or 8 feet per second: with clear water in starting or washing out pipes, up to double these amounts; with load of heavy sand lodged in the bottom of the pipe or with accumulations tending to block or actually blocking pipe, varying down to 0 feet per second.

The material is described as varying "all the way from soft with very fine sand, to almost pure sand of a very heavy variety, and included considerable quantities of a very stiff, sticky clay."

The percentage of solids was figured as about 15 on the average

> . . . though with pumps in good shape and load not too heavy, it is fair to state that we have gotten for considerable periods over 20 per cent. In stiff, sticky clay occasionally we have gotten a practically solid mass at the discharge, this, of course, being an exceptional case when there was danger of plugging the pipe. . . . Velocities undoubtedly vary in different sections of the pipe, due to the pipe filling with heavy sand, some of which may be stationary, but most of which is moving at a much lower velocity than the lighter stream above . . . especially in the section astern of the dredge. Pipes that have been separated by the bursting of a sleeve or otherwise have been found in exceptional cases nearly half full of heavy sand, though ordinarily there would be only three or four inches in the deposit. . . . The layers of sand encountered here, helped

in preventing an accumulation, many balls coming out with a good coat of sand. . . .

The clogging of the pipe was practically entirely caused by the stiff clay and its accumulation in masses as mentioned above. Sand accumulations that might have lead to clogging were detected by the gauge readings and readily reduced by pumping clear water. . . .

The friction head per thousand feet of pipe varied between wide limits, but 25 feet was taken as the average and used in calculations by means of which to locate relays. For water only the friction head due to a 7-foot velocity should be about 10 feet per 1,000 and that due to an 8-foot velocity should be about 13 feet per 1,000. A friction head of 25 feet per thousand would correspond to a velocity of about 11.5 feet per second.

. . . With few exceptions the friction head is the greatest in the dredge discharge pipe, in some measure due to the naturally higher friction in floating pipe compared with shore pipe (estimated at about 25 per cent), but mainly to the heavier deposit of sand in this section compared with the sections beyond the relays. . . .

This data is exceedingly interesting for it is evidently a limiting case. In the section of pipe nearest the dredge a sudden heavy charge of sand produced a deposit. The losses due to increased friction were noted. This deposit was in the nature of an evener for the remainder of the pipe, and with an average of 15 per cent of solids there was little trouble beyond the first relay. It is evident that at times the current near the dredge was overloaded. In view of the fact that there were two other pumps to be considered, it was probably wise to suffer losses in the first section of pipe in order to obtain a maximum average duty in the second and third sections.

In a paper, "Hydraulic Dredging on the Mississippi River," by F. B. Maltby, Member of the American Society of Civil Engineers, and published in *Transactions American Society Civil Engineers*, Vol. LIV, Part C, measurements showing the per cent of sand in various portions of a dredge pipe are given. The measurements were taken by means of a small pipe passing through a stuffing box into the discharge pipe. At the lower end the small pipe was bent to a right angle with the vertical stem. The small pipe could be shoved up or down and the opening at the lower end could be turned to face the current. Samples of the mixture were thus obtained along points of a vertical diameter. Mr. Maltby was not

successful in obtaining simultaneous velocity measurements. Mr. Maltby concludes:

The evidence that the whole cross-section of the discharge pipe is effective is quite conclusive, as in no case was there any evidence that any sand was resting on the bottom of the pipe, . . .

The table below gives the result of measurements on the *Kappa*. The writer believes that the second column shows a case of concentration near the bottom very similar to the description on page 226 of the action of sand in an open channel at the moment that the velocity has reached a point slightly higher than the velocity at which sand waves disappear. When pumping water only the velocities in the *Kappa's* pipe were from 19.7 to 22.1 feet per second with 240 feet of ponton pipe. With the *Henry Flad*, practically an identical dredge, the velocities, when pumping water only, ranged from 14.8 to 19.1 feet with 480 feet of ponton pipe. There is no data given by means of which the actual velocity while the observations were being made can be approximated.

Table No. 38, p. 459, Vol. LIV, Part C, *Transactions American Society Civil Engineers*, percentage of sand handled by *Kappa*, Presidents Island, November 28th and 29th, 1902:

Distance from top of pipe, in inches.	1	2	3	4	5	6	7	8	Means
Top	0.031	0.067							
2	0.281	0.193	0.125	0.125	0.088	0.100	0.063	0.030	0.126
4	0.255	0.080	0.140	0.155	0.125	0.100	0.063	0.063	0.123
6	0.271	0.099	0.125	0.105	0.125	0.125	0.083	0.188	0.140
8	0.207	0.118	0.155	0.188	0.150	0.250	0.125	0.188	0.172
10	0.247	0.129	0.155	0.220	0.125	0.125	0.125	0.168	0.162
12	0.050	0.147	0.188	0.220	0.188	0.168	0.083	0.125	0.146
14	0.110	0.135	0.188	0.235	0.214	0.168	0.188	0.088	0.166
16	0.073	0.152	0.155	0.205	0.175	0.188	0.167	0.125	0.155
18	0.214	0.125	0.250	0.220	0.171	0.205	0.083	0.168	0.167
20	0.147	0.119	0.188	0.313	0.168	0.250	0.083	0.125	0.174
22	0.131	0.138	0.188	0.280	0.168	0.168	0.125	0.168	0.171
24	0.177	0.214	0.280	0.250	0.188	0.125	0.125	0.125	0.185
26	0.130	0.193	0.280	0.225	0.179	0.125	0.125	0.188	0.181
28	0.137	0.339		0.219	0.174	0.168	0.125	0.125	0.198
30	0.171	0.467		0.250	0.168	0.125	0.125	0.125	0.204
Bottom 32				0.250	0.250	0.188	0.250	0.250	0.238
Means	0.165	0.169	0.186	0.216	0.168	0.161	0.121	0.141	0.169

The material was ordinary channel sand, possibly slightly finer than the average, pumped through 240 feet of 32-inch discharge pipe.

The measurements were made at the center of the second ponton.

VII. FRICTION LOSSES DUE TO FLOW OF WATER THROUGH DREDGE PIPES.

Many formulæ have been advocated for determining the friction losses due to the flow of water through pipes. Until quite recently most of these formulæ assumed that the velocity varied with the square root of the sine of the angle of slope, with the square root of the diameter of the pipe and with a factor depending upon the roughness of the pipe. Of these, the Chezy formula was the simplest in form, $V=C\sqrt{\frac{D}{4}\times S}$. Another form known as the Darcy formula is $F=\frac{H\times 2g\ D}{L\times V^2}$. In the Chezy formula, C, and in the Darcy formula, F, were called the friction factors and they were originally supposed to be constants.

Experiments showed that C and F in the above formulæ were not constants, but in reality varied with the slope and with the diameter of the pipe. Kutter's formula, a very awkward formula to use, was very largely utilized for determining the value of C in the Chezy formula for various slopes and diameters after a new coefficient of roughness, n, had been determined.

In the various editions of Trautwine these and other formulæ will be found, together with tables to assist in finding the coefficient of roughness.

The actual experiments indicated that if an exponent slightly higher than 0.5 be assigned to D and S in the Chezy formula, the coefficient C would become a constant within the range of ordinary practice and a formula:

$$V=C\times R^{0.63}\times S^{0.54}\times 0.001^{-0.04}$$

was suggested by Prof. Gardner S. Williams and Mr. Allen Hazen, C. E. A diagram for the use of this formula will be found in the 1913 edition of Trautwine. Published tables and a special slide rule based on this formula are also in existence.

If the coefficient of roughness C for a specific dredge pipe be once determined for the above formula, then no further variations in C need be anticipated. The use of tables, etc., to determine new values of C for each change in slope is unnecessary. The writer has seen a few determinations of the coefficient of roughness in floating dredge pipes and in all his calculations the value C=110 is used. This value corresponds to the value given in Trautwine for cast iron pipe ten years old. For well laid shore pipe a higher value of C should probably be used. In such cases the value might run as high as C=120.

A diagram for 16, 20, 24, 27 and 30-inch pipes, based on the Williams-Hazen formula for C=110, is given in Fig 5.

VIII. FRICTION LOSSES DUE TO THE FLOW OF MIXTURES OF WATER AND SUSPENDED MATTER.

The causes of suspension of solids have been discussed at some length. Let us now endeavor to learn something of the energy expended in dredge pipes due to the presence of this load of suspended matter.

The force of gravity is always present. If the numbers, sizes and specific gravities of the particles in suspension are known we can from Mr. Thoulet's table, page 16, determine the work they are capable of performing if allowed to fall freely through water. A sufficient amount of energy must be imparted to the current to overcome this tendency. To obtain some idea as to the extent of this loss a hypothetical case has been assumed, but it is made to compare approximately with what may be encountered in practice.

On page 1383, *Annual Report Chief of Engineers, U. S. Army,* 1878, a description of a boring made at the foot of Presidents Island is given. Among the materials encountered were beds of sand, which are described as follows:

Sand: 25 per cent less than .01 inch; 50 per cent between .01 and .02 inch; 25 per cent over .02 and less than .03 inch.

Sand: 50 per cent between .02 and .03 inch; 40 per cent between .01 and .02 inch; 10 per cent under .01 inch.

Sand: 20 per cent over .03 inch; 60 per cent between .01 and .03 inch; 20 per cent less than .01 inch.

Sand: of uniform size between .01 and .02 inch; 10 per cent between .02 and .03 inch.

Sand: 10 per cent less than .01 inch; 20 per cent between .01 and .02 inch; 60 per cent between .02 and .03 inch; 10 per cent between .03 and .04 inch.

Same as above with a few pebbles about 0.2 inch across.

Sand: 10 per cent under .01 inch; 10 per cent between .01 and .02 inch; 40 per cent between .02 and .04 inch; 40 per cent between .04 and .10 inch.

Coarse sand the size of grains increasing toward the bottom; not more than 10 per cent being under .02 inch; 30 per cent between .02 and .08 inch; 60 per cent between .08 and .15 inch.

Since .01 inch is equal to .25 mm., the various sizes of the hypothetical sand are assumed to occur in the following percentages:

Diameter in MM.	0.2	0.4	0.6	0.8	1.0
Per cent	10	25	30	30	5

The specific gravity is assumed to be 2.5.

In the following table, column 2 gives the velocity in feet per second at which the particle would fall through still water, assuming this velocity to be 0.7 of the velocity determined for spherical particles. Column 3 gives the work in foot-pounds that would be done by 100 pounds of sand grains of the particular size in falling through a distance equal to their velocity per second. Since the particles are submerged, the effective weight is reduced to the extent of the weight of the water displaced. Column 5 is the product of columns 3 and 4 and its total represents the work that would be done in 1 second by 100 pounds of the hypothetical sand.

Diam. MM.	V feet per sec.	Work 100 lbs. each size.	Per cent each size	Product Col. 3×Col. 4
1	2	3	4	5
0.2	.071	4.26	10	.426
0.4	.123	7.38	25	1.845
0.6	.167	10.02	30	3.006
0.8	.203	12.18	30	3.654
1.0	.235	14.10	5	.705
				9.636

In 1,000 feet of 30-inch pipe carrying 25 per cent of sand the total weight of sand would be 98,180 pounds, and to keep it in suspension would require 9,450 foot-pounds of work per second, or 17.2 horsepower.

From an examination of the data in regard to tests of Mississippi River dredges, which will be discussed later, the writer concludes that the horsepower expended in 1,000 feet of 30-inch dredge pipe when discharging 25 per cent of river sand will average fully 90 horsepower more than when pumping water only at the same velocity. This would indicate that the loss due to the above cause is barely one-fifth of the total loss.

Mr. Thoulet's table shows that for a given expenditure of energy a greater weight of finely divided solids may be held in suspension than if the same weight be concentrated in a less number of par-

ticles. Take, for example, a sphere 5 mm. in diameter and 2.5 specific gravity. To hold it in suspension for 1 second it will be necessary to expend an amount of energy represented by its weight in water times its velocity, 206 mm. per second. If this sphere be replaced by 125 spheres of 1 mm. diameter each, and of the same density, the combined weight of the 125 small spheres will equal that of the single large sphere. The energy required to keep them in suspension for 1 second will be their weight in water times their velocity 107 mm. per second. The amount of energy required is only 52 per cent of what was required in the first place.

A current of water flowing through a horizontal pipe and carrying a single grain of sand must have imparted to it a greater amount of energy than would be required if that particle of sand were not present. Two losses due to the presence of this particle are:

1. In each second of time enough energy must be applied to equal the work that the particle of sand would do in falling the distance indicated in Mr. Thoulet's table. This loss may be readily computed if the sizes, specific gravities, and number of particles are known.
2. Each time a change of direction is made in the path of the particle energy will be wasted in effecting this change in direction. From the very nature of the case this can not be determined in advance for any specific pipe line.

The former of these losses is apparently only a small part of the whole. It is independent of the velocity of flow; it is independent of the roughness of the pipe; it increases directly with the number of particles present; it increases rapidly with the size and specific gravity of the particles; for finely divided clay and mud it is negligible.

The second loss is less well understood. It probably increases with the square of the velocity, with the roughness of the pipe, and with the size and specific gravity of the particles.

A third loss, and probably one of the most important, is the rolling and sliding friction of particles too heavy to be held in full suspension. This loss decreases with each increase in velocity.

The theory of these frictional losses is so complex that the writer will not attempt to discuss it further, but will turn to such experimental data as he has been able to find and will endeavor to deduce an empirical rule that may be used until further experiments have thrown more light upon the subject.

On pages 400 to 408, *Transactions American Society Civil Engineers*, Vol. LVII, Miss Nora Stanton Blatch, Junior American Society of Civil Engineers, gives the results of some experiments with mixtures of sand and water flowing through 1-inch brass and 1-inch iron pipe. The curves of one of her diagrams are shown in Fig. 4.* This set of experiments covered a wide rage of velocities and per cent of solids. The author knows of no other experiments that have been carried as far as these. The sand used in the experiments upon which Fig. 4 is based had a specific gravity of 2.64 and lay between sieves of 20 and 40 mesh. To observe the character of flow short sections of glass tubing were inserted in the

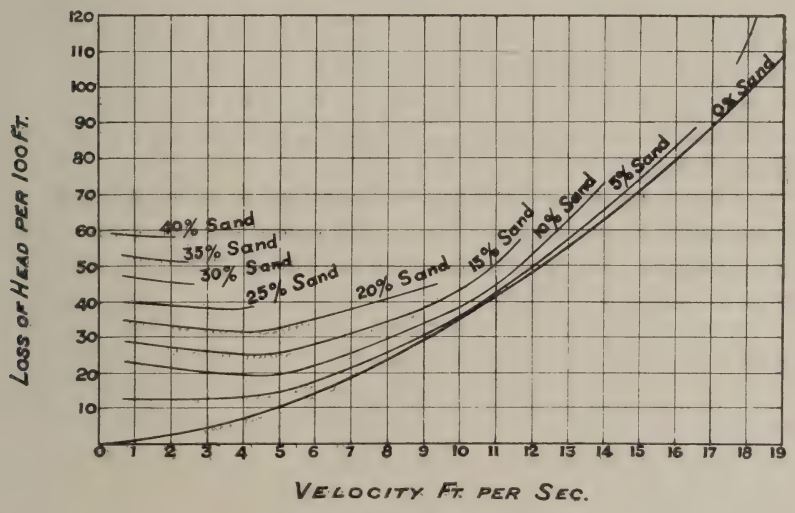

Fig. 4.

system up and down stream from the lengths tested. Miss Blatch states, "It was found that, for velocities of less than about 3½ feet per second, the greater part of the sand was dragged along the bottom of the pipe at a very low velocity, practically clear water flowing in the upper part of the cross-section. At a velocity of about 3½ feet per second large quantities of sand began to be carried in suspension . . . This change occurred suddenly for the fine grade of sand (held between 60 and 100 mesh sieves) while for the coarse grade, in which the particles differed more in

* Reproduced by permission of the American Society of Civil Engineers from data given on page 401, Vol. LVII, *Transactions American Society of Civil Engineers.*

size, the change was gradual. At the higher velocities, all of the sand was carried in suspension throughout the cross-section."

Referring to the diagram it will be seen that for 10 per cent or more of sand an increase in velocity produced a decrease in loss of head until a velocity greater than 4 feet had been reached. The writer ventures the opinion that for velocities less than about 5 feet the forces tending to throw the particles away from the sides, and particularly from the bottom of the pipe, were unequal to their work,and that rolling and sliding friction as well as sand waves and restricted water ways were present. The glass tubes used were "slightly less than 1 inch diameter, the brass tube had an actual diameter of 1.0546." The glass sections were connected to the other piping by rubber tubes and eddy currents should have been especially strong in their neighborhood. These facts may account for the fact that in the glass tubes partial suspension occurred before the loss of head diagram shows any change in condition of flow.

For velocities of about 5 feet to velocities of about 9.5 feet the curves approximate straight lines approaching the curve for water. For velocities greater than 9.5 feet the curves gradually separate from the curve for water.

For practical dredging operations the condition of flow between the 5 and 10 foot velocities deserves the closest scrutiny. For any fixed percentage of sand any increase in velocity between these limits means a loss in power, but the loss is less rapid than it would be if pumping water only. For any fixed velocity the loss due to the addition of 1 per cent of sand increases to some extent with the per cent of sand already in suspension.

Miss Blatch's diagram for iron pipe, actual diameter 1.0420 inch, indicates that the sand became largely suspended when the velocity reached about 3.5 feet per second. For velocities of 1 foot to 3.5 feet per second the loss of head for any particular percentage of sand was a constant. The curve for 15 per cent of sand begins to diverge from the curve for water only at a velocity of about 8 feet. The total friction loss is greater than for brass pipe. The principal differences from a practical standpoint are that increased roughness caused the sand to become suspended at a lesser velocity; that the range in the velocities during which increased velocity resulted in a smaller increase in frictional losses than for water only, remained about the same; that is, in the case of the iron pipe, the change occurred about 1 foot earlier at both

the lower and upper limits for this condition of flow; and that the total frictional loss for the rough pipe is greater under all conditions than for the smooth pipe.

Miss Blatch's experiments with fine sand, retained between 60 and 100-mesh sieves, indicated that the sand became all suspended at a velocity of about 3.0 feet, and that for velocities greater than 4 feet the loss of head for mixtures of sand and water increased more rapidly than the loss of head for water only. For velocities greater than about 7.25 feet the loss of head for 10 per cent of fine sand was greater than for 10 per cent of coarse sand. The writer is unable to account for this.

The writer ventures the suggestion that with the brass pipe the coarse sand became entirely suspended at about 9.5 feet and with the iron pipe at about 8.5 feet. For velocities less than this, rolling and sliding friction were to some extent present.

The diagrams prepared by Messrs. Hazen and Hardy and published on pages 345 and 347 of the same volume, indicated that for 3-inch and 4-inch pipe the curves for sand mixtures approach the curves for water for velocities from 3 to 10 feet per second.

These are the only published experiments, known to the writer, which were made with the express purpose of determining the frictional losses due to the flow of mixtures of sand and water in pipes. In many other tests with dredges, etc., the total frictional loss and percentage of sand are given, but the coefficients for flowing water through the same pipes under the same conditions are not given. As will be seen later, calculations based on this data are inconsistent in many cases. This is to be expected, as it was recorded as a mere incident to tests conducted for an entirely different purpose. Changes which might vitally affect our conclusions were not recorded because they had no bearing upon the questions at issue at the time the tests were made.

Miss Blatch's experiments may be sufficient to establish a law for frictional losses due to the flow of mixtures of sand and water in brass pipes of 1-inch diameter. Her data also furnishes coefficients for the particular size of pipe used by her. The general trend of all data known to the writer points to the conclusion that the form of curve established by Miss Blatch for 1-inch brass pipe applies to dredge pipes up to 34-inch diameter. The coefficients are, however, very different. In view of the rapidly changing conditions in all dredging operations, the writer believes it to be more important to establish a simple formula that will give safe results

than to establish a complicated formula by means of which exact results might be obtained if the material were uniform and its properties exactly known.

For practical operations the writer believes that the condition of flow should be that obtaining between the 5 and 10-foot velocities during Miss Blatch's experiments. Had she mixed the two grades of sand used, the tendency on the part of the curves for coarse sand to approach the curve for water would have been to some extent counteracted by the tendency on the part of the curves for fine sand to separate from the curve for water only. The writer assumes in his formula that the curves will be parallel. If the numerical coefficients are taken from the condition prevailing at the 5-foot velocity in Miss Blatch's experiments, they will, if applied to a somewhat higher velocity, give a factor of safety.

The writer's tentative formula assumes that the additional frictional loss due to the addition of 1 per cent of sand is a constant, no matter how much sand is already in suspension. If this coefficient is obtained by dividing the loss due to the highest percentage handled by the percentage, it will give a factor of safety for any lower percentage.*

The formula becomes:

$$H = H_w + H_s = H_w + A\,N$$

In which H=the total loss of head due to the flow of the mixture of sand and water,

H_w=the loss of head due to the flow of water only,
H_s=the excess loss of head due to the presence of sand,
A=a factor depending upon the character of the sand,
N=the percentage of sand present.

The writer obtains his value of H_w by means of the Hazen-Williams slide rule. In an article, "Hydraulic Dredging on the Mississippi River," by F. B. Maltby, Member American Society Civil Engineers, *Transactions American Society Civil Engineers,* Vol. LIV, Part C, p. 456, diagrams showing the loss of head in the discharge pipes of various Mississippi River dredges are given.

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* After the above was written the writer's attention was called to the following statement in an article for the *American Civil Engineers' Pocket Book,* by Mr. Allen Hazen, C. E. (p. 935, 1912 edition).

"For friction of sand and water in 3 and 4-inch pipes, compute the friction for water alone, and add 3.5 feet per thousand for each per cent of sand in the mixture. For 6-inch or larger pipe add 2.5 feet and 2.5-inch hose, add 4.5 feet per thousand. These figures will be close enough for velocities 5 feet per second or over. Sand and water mixtures will flow well at all velocities above 5 feet per second and fairly well from 4 to 5 feet per second. Between 3 and 4 feet per second there will be more friction than calculated and some stoppages; and below 3 feet per second sand and water mixtures will not flow."

A glance at these diagrams shows how a bend in the line, an ill-fitting sleeve, etc., may seriously affect the hydraulic gradient. On page 454, Mr. Maltby gives results on a straight section of the *Beta's* pipe line. The corresponding value of C in the Hazen-Williams formula is 110. This value is used by the writer for dredge pipes unless there is some special reason for making an exception, in which case the exception is explained in a note. The value of A computed by the writer should be somewhat too large, as it compensates for irregularities in the pipe line. Any error introduced by this fact is on the side of safety.

From Miss Blatch's diagrams the following values of A are found:

1-inch brass pipe, sand specific gravity, 2.64, V=5 feet per second.

Sand retained between 20 and 40-mesh sieves	N=20	A=11.0
Sand retained between 20 and 40-mesh sieves	N=10	A= 9.0
Sand retained between 60 and 100-mesh sieves	N=10	A= 6.0

1-inch galvanized iron pipe, other conditions as above,

Sand retained between 20 and 40-mesh sieves	N=20	A=11.0
Sand retained between 20 and 40-mesh sieves	N=10	A= 8.0

From the diagrams of Messrs. Hazen & Hardy the following values are obtained after assuming a value C=120 for the pipe.

The sand is understood to have been Potomac River sand. V=5.

3-inch iron pipe	N=25	A=3.4
	N=10	A=3.9
4-inch iron pipe	N=25	A=3.2
	N=10	A=3.6

The following table gives results obtained by Mr. J. A. Vogleson, Member American Society Civil Engineers, at the Shawmont Pumping Station, Philadelphia, Pa., 1901, page 391 *Transactions American Society Civil Engineers,* Vol. LVII. The pipe was 4-inch iron pipe laid horizontally. The sand was used over and over again. C was assumed as 120.

4-inch Pipe.

From Mr. Vogleson's table. Computed by the writer.

Velocity ft. per sec.	Per cent of sand.	Slope.	H	Hw	Hs	A
4.12	6.1	.0354	35.4	21.1	14.3	2.35
4.55	13.8	.0589	58.9	25.2	33.7	2.44
4.64	16.6	.0648	64.8	26.3	38.5	2.32
4.58	15.1	.0648	64.8	25.8	39.0	2.58
4.19	11.2	.0530	53.0	21.8	31.2	2.78
4.59	17.1	.0589	58.9	25.9	33.0	1.93
4.53	16.3	.0589	58.9	25.1	23.8	1.46
4.45	15.9	.0530	53.0	24.3	28.7	1.80
4.63	17.6	.0530	53.0	26.2	26.8	1.52
4.48	16.2	.0530	53.0	24.6	28.4	1.75
4.72	18.3	.0589	58.9	27.1	31.8	1.74
4.46	14.0	.0530	53.0	24.5	28.5	2.04
4.54	14.6	.0530	53.0	25.1	27.9	1.91
4.81	19.6	.0589	58.9	28.1	30.8	1.57
4.91	16.1	.0589	58.9	29.2	29.7	1.84
					Total	30.03
					Mean	2.00

Test of United States dredge *Alpha.* See Annual Reports of Chief of Engineers, 1895, page 3796; and 1898, page 3342. Also *Transactions American Society Civil Engineers,* Vol. LIV, Part C. Diameter of pipe, 30 inches. C=110.

From published tables. Computed by the writer.

Test No.	H′	V	N	H	Hw	Hs	A
1895.	*Length of pipe, 604.5 feet.*						
7	15.53	13.31	5.63	25.8	20.8	5.0	0.89
8	15.80	11.10	12.45	26.2	14.8	11.4	0.92
9	15.39	11.74	10.89	25.5	16.4	9.1	0.84
10	15.38	11.74	7.61	25.5	16.4	9.1	1.20
11	16.59	12.41	10.27	27.4	18.3	9.1	0.89
12	15.23	12.04	6.37	25.3	17.2	8.1	1.27
13	14.08	12.85	9.21	23.3	19.3	4.0	.43
14	15.90	11.48	8.67	26.3	15.8	10.5	1.21
15	15.32	12.94	6.20	25.4	19.6	5.8	0.94
	Length of pipe, 1,059.5 feet.						
16	22.13	9.78	5.23[1]				
17	22.80	7.53	7.38[2]				
18	22.64	7.85	12.17	21.4	7.8	13.6	1.12
19	23.53	9.20	9.11	22.2	10.4	11.8	1.30
20	25.52	8.94	6.93	24.1	9.9	14.2	2.05
21	23.36	9.46	7.75	22.0	11.0	11.0	1.42
22	20.73	10.79	8.90	19.6	14.1	5.5	0.62

Test No.	H′	V	N	H	Hw	Hs	A
1898.	*Length of pipe, 1,131 feet.*						
3	35.0	9.6	14.0	30.9	11.3	19.6	1.40
4	35.0	8.6	4.0[3]				
5	32.7	11.4	12.0	28.9	15.6	13.3	1.11
6	24.6	12.4	0.0	21.8	18.2		
7	53.4	7.2	25.0[4]				
8	38.5	3.9	21.0[5]				
9	28.1	9.7	14.0[6]				
	Length of pipe, 599 feet.						
1	25.8	14.2	0.0[7]	43.1	23.2		
2	30.4	13.9	18.0[7]	50.7		7.6	.42
10	30.4	14.1	16.0[7]	50.7		7.6	.48

[1] Mud in sand.
[2] Clay lumps in sand.
[3] One minute after this test the suction was found to be clogged.
[4] Pressure gauge clogged.
[5] Pipes purposely overloaded.
[6] Pressure gauge clogged.
[7] In these three tests the velocities were nearly equal and in one of them water only was pumped. The difference between the loss of head when pumping sand and when pumping water is assumed to be the excess due to sand.

Test of United States dredge *Gamma.* See Annual Report Chief of Engineers, 1898, page 3327, and *Transactions American Society Civil Engineers,* Vol. LIV, Part C.

Diameter of pipe, 34 inches; length, 1,069 feet. C=110. 1897.

From published tables. Computed by the writer.

Test No.	H′	V	N	H	Hw	Hs	A
1	29.3	11.59	14.3	27.4	13.8	13.6	0.95
2	35.0	10.64	16.0	32.7	11.9	20.8	1.30
3	38.5	8.48	15.8	36.0	7.8	28.2	1.78
4	30.9	11.37	8.1	28.9	13.4	15.5	1.92
5	35.0	10.57	22.2	32.7	11.7	21.0	0.95
6	38.7	9.79	21.6	36.2	10.2	26.0	1.20
7	41.5	7.84	21.4	38.8	16.7	22.1	1.03
8	41.5	9.42	27.6	38.8	9.4	29.4	1.06
9	33.9	10.77	13.2	31.7	12.1	19.6	1.49
10	45.4	5.15	21.0[1]				
11	37.3	10.64	29.0	34.9	11.8	23.1	0.80
12	30.4	12.99	6.6	28.4	17.1	11.3	1.71
						Total	14.19
						Mean	1.29

[1] Pipes sunk at test.

Test of United States dredge *Epsilon*. See Annual Report Chief of Engineers, 1898, page 3332, and *Transactions American Society Civil Engineers*, Vol. LIV, Part C.

Diameter of pipe, 32 inches; length, 1,121 feet. C=110. 1898.

From published tables. Computed by the writer.

Test No.	H′	V	N	H	Hw	Hs	A
1	44.7	10.2	9.9[1]				
2	34.7	14.7	6.5[1]				
3	34.7	16.9	13.6	30.9	29.9	1.0	0.07
4	37.7	15.7	24.6	33.6	26.2	7.4	0.30
5	37.7	16.4	25.8	33.6	28.3	5.3	0.21
6	37.7	16.6	22.9	33.6	29.0	4.6	0.20
7	36.7	18.3	16.5	32.7	34.7	2.0	
						Total	0.78
						Mean	0.19

[1] Mud lumps found in barge.

Test of United States dredge *Delta*. See Annual Report Chief of Engineers, 1898, page 3329, and *Transactions American Society Civil Engineers*, Vol. LIV, Part C.

Diameter of pipe, 34 inches; length, 1,104 feet. C=110.

From published tables. Computed by the writer.

Test No.	H′	V	N	H	Hw	Hs	A
1	46.4	16.19	12.8	41.9	25.7	16.2	1.27
2	45.3	16.50	11.7	41.0	26.8	14.2	1.21
3	45.3	15.59	17.0	41.0	24.0	17.0	1.00
4	54.5	12.09	17.8[1]				
5	47.6	14.96	7.4	43.1	22.2	20.9	2.82
6	45.3	14.71	19.6[2]				
7	47.6	14.01	25.1[3]				
8	47.6	15.41	7.7[2]				
9	38.4	13.92	18.0	34.7	19.4	15.3	0.85
10	43.0	15.01	25.5	38.9	22.4	16.5	.65
11	41.8	15.39	14.3	37.8	23.4	14.4	1.01
12	43.0	14.68	11.9	38.9	21.5	17.4	1.46
13	45.3	14.80	10.0[4]	41.0	21.8	19.2	1.92
14	46.4	15.10	7.9[4]	42.0	22.7	19.3	2.44
15	33.8	18.45	9.9[5]	63.6	32.8	30.8	3.11
16	38.4	15.90	9.5	34.8	24.9	9.9	1.04
						Total	8.49
						Mean	1.06

[1] Not included in average.
[2] Test barge valve not completely open.
[3] Cutter broke at test.
[4] Sand with small per cent of gravel not included in average.
[5] Discharge pipe 532 feet long not included in average.

The preceding tables illustrate the well-known fact that the conditions under which a dredge is working vary between wide limits. If a mistake has been made in assuming too small a value of C, then the coefficient A is too small. The contrary is, however, believed to be the case and for large dredges handling sand the value A=1.0 is probably safe.

So far as actual results are concerned the following are of interest:

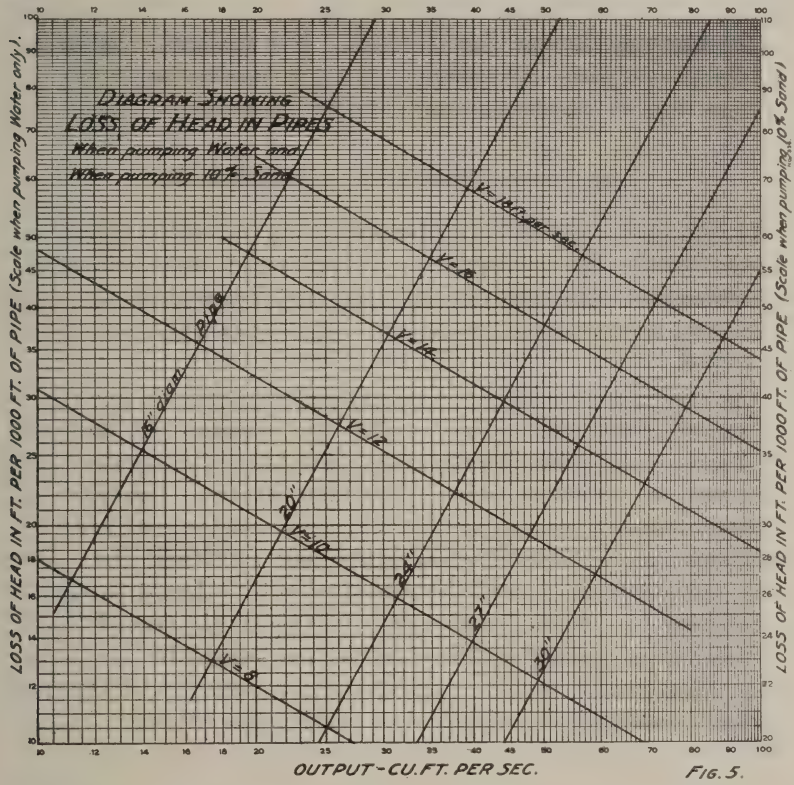

FIG. 5.

(a) A dredge working at Lock Raven in connection with the Baltimore water supply discharge material through 9,165 feet of 22-inch pipe and 1,632 feet of 18-inch pipe with a pressure of 28 pounds and a negative head of 10 feet. Assuming friction losses due to water only, this would indicate a velocity of about 5.3 feet per second in the 22-inch pipe. The material handled was sediment deposited in an artificial reservoir by storm water from a cultivated water shed. It is described as "fine sharp sand and silt in almost equal proportions with some stones, but no clay or gravel." There was some trouble with discharge pipes clogging.

(b) A representative of the Atlantic, Gulf and Pacific Company stated, as a result of his experience with a certain 20-inch dredge, that when working in mud or fine sand a pressure of 42 pounds corresponded to about 3,500 feet of discharge pipe (about 26 feet per M.); when working in "gravel formation with gravel up to the size of one's fist" the same pressure corresponded to an 1,800-foot pipe line (about 50 feet per M.).

The preceding diagram, Fig. 5, has been prepared to show the friction losses in dredge pipes due to the flow of water only or water carrying 10 per cent of sand. For the great majority of dredging operations the actual results will show losses intermediate between the two extremes shown on that diagram.

IX. POWER EXPENDED IN DREDGE PIPES.

If Fig. 5 be accepted as representing the probable friction losses due to the flow of a mixture of 10 per cent of sand and water, a corresponding diagram showing the power required to operate 1,000 feet of pipe line may be prepared. Such a diagram is given in Fig. 6.

The advantage of using the largest diameter of pipe line that can be operated with the power available is obvious. In the normal case we are so limited by circumstances that only one size of pipe line can be considered for a particular dredge. Occasions may, however, arise, where a short 24-inch line could be used with great advantage on a dredge that normally uses a 20-inch line. Whenever relays are to be used the size of pipe line should be carefully considered. A 20-inch dredge may be able to dig enough material to warrant a capacity of 35 cubic feet per second in the discharge pipe. This would maintain an adequate velocity in a 24-inch pipe and the power of the relay pump could be so adjusted that a 24-inch line would be the proper size to use.

From Fig. 5 the loss of head for a given condition may be obtained. There is a certain maximum pressure at the stern of a dredge that should not be exceeded. The cost of sleeves for high pressures is very great.

If 10 feet be assumed as the minimum velocity and if 150 and 100

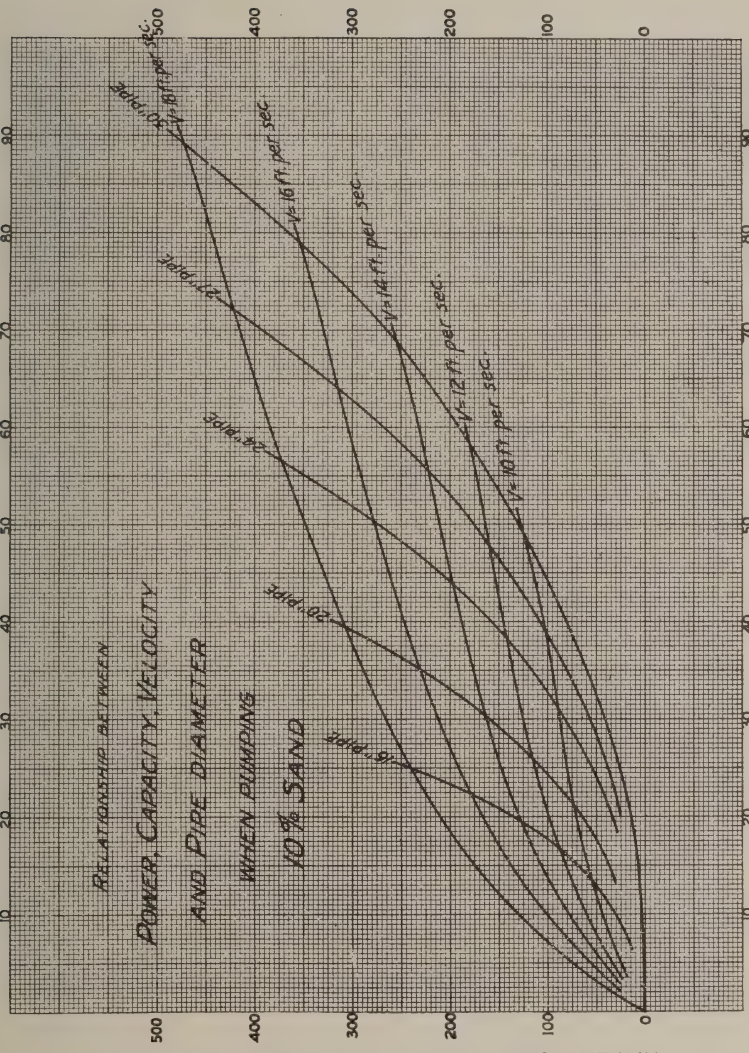
Relationship between
Power, Capacity, Velocity
and Pipe Diameter
when pumping
10% Sand
Water horse power per 1000 ft. of pipe.
Output - cu. ft. per sec.
20 ft. per sec.
16 ft. per sec.
12 ft. per sec.
10 ft. per sec.
8 ft. per sec.
6 ft. per sec.
4 ft. per sec.
3 ft. per sec.
2 ft. per sec.
0
100
200
300
400
500
10
20
30
40
50
60
70
80
90

Fig. 6.

feet be assumed as maximum heads, the maximum length of pipe line and the corresponding water horsepower becomes:

Diam. of pipe.	150-foot head.		100-foot head.	
	Max. length of pipe.	Water horse-power.	Max. length of of pipe.	Water horse-power.
Inches.	*Feet.*		*Feet.*	
16	4240	246	2830	164
20	5070	380	3380	253
24	5810	523	3870	348
27	6300	662	4200	441
30	6760	852	4500	568

Dredges having the above power available and working in mud could operate with considerably longer pipe lines.

X. POSTSCRIPT.

Referring to the seven original questions, it will be seen that (b), (c) and (d) have not been touched. It is hoped to take these up in a subsequent paper.

Approximate rules for answering the remaining questions have been given. Taking the San Juan project as an illustration, we find:

1. As the discharge pipes should be long and as the quantities to be excavated are sufficient to warrant the use of a large plant, a 27-inch pipe line should be considered a minimum.

2. The "soft" material will, it is believed, readily subdivide into small particles resulting in a thin muddy mixture of quite even consistency. The velocity required to carry this mixture does not in all probability exceed 8 feet per second. The frictional losses will be intermediate between those shown by the two scales of Fig. 5. An increase of 5 feet per 1,000 feet of pipe line is considered ample.

3. The "tough" material consists of various grades of clay. It is believed that the cutter will slice this off in chunks. Many chunks will be rounded during the process of transportation and will appear on the spoil bank in the form of clay balls. A low velocity will transport individual balls of this material, but masses of this nature have a habit of coalescing and forming larger masses. If a good velocity be not maintained there is serious danger of clogging. A velocity of 10 feet per second is considered advisable.

4. The "tough" material also contains a small percentage of "coral débris." This consists of particles $\frac{5}{8}$ inch in diameter and 5 inches long and smaller sizes. The specific gravity of the coral is not high and the porous structure results in a relatively large surface exposed to the action of the current as compared to the weight of an individual particle. The shape of the particles is not favorable for transportation by rolling. For transportation of this ma-

terial a velocity of at least 10 feet per second is considered advisable.

5. With a 10-foot velocity, 20 or 25 per cent of "soft" materials should be readily transported. With the "tough" materials such high percentages would probably make trouble. Eleven and one-half per cent of solids with a 10-foot velocity in a 27-inch pipe represents approximately 600 cubic yards of solids per hour. It is believed that 11½ per cent of solids would be safe so far as the ability of the current to transport them is concerned. If a dredge can loosen as much as 600 cubic yards per hour of the tough material, it is believed that the operators will be well satisfied. For all except the longest pipe lines a higher velocity will be available.

6. With a 10-foot velocity, and the assumption made in par. 2 above, the loss of head per 1,000 feet of pipe becomes 18.8 feet. For a maximum head of 150 feet at the stern of the dredge approximately 8,000 feet of pipe line is the limit. The water horse power expended in the discharge pipe is 657.

Note.

The maps of the battlefields of Franklin and Nashville appearing on pages 195 and 201 of this number were reproduced, by permission of the Century Company, from Vol. IV of *Battles and Leaders of the Civil War.*

Erratum.

The references in paragraph 16, page 36, and paragraph 56, page 58, of No. 31, PROFESSIONAL MEMOIRS, should be to *James B. Francis*, not to *David M. Francis.*

Book Reviews.

THE REGULATION OF RIVERS. By J. L. Van Ornum, C. E., M. Am. Soc. C. E. First Edition. 1914. 379 pages, 6 by 9 inches, colth. Illustrated (4 plates and 99 figures). McGraw-Hill Book Company, New York. Price, $4.00 net.

This volume commends itself in several ways to those interested in river improvement by regulation and in levee work. It is written by one who as an Assistant Engineer in the Government service has had first hand opportunity to gather an intimate knowledge of river works. His status of consulting engineer and that of professor of civil engineering at Washington University enable him to maintain an authoritative position on the subjects, and the latter capacity (that of a teacher in a responsible position) fits him to present his views and data in a form easily assimilated by the student engineer as well as in a manner interesting to the graduate engineer.

Another point of advantage to this work is its lack of company in its particular field, which is apparent to anyone who has sought for literature on modern river improvement, particularly American books dealing with American rivers. The author does not, however, confine himself to American works.

The book is written in good English, except that the sentences seem unduly long; still, they are clear and meaty. The illustrations are well chosen; they could be more numerous, perhaps, and some of the halftones, if more distinct, would have enhanced the appearance of the volume.

The chapters are ten in number and are headed:

Commercial Considerations,
General Phenomena,
Investigations, Surveys, etc.,
Methods of River Improvement,
The Principles of Regulation,
Works of Channel Contraction,
The Protection of Erodible Banks,
Dredging,
Levees,
The Control of the Current.

Naturally, in the limit of 379 pages, none of the subjects is treated exhaustively. The chapter on Commercial Considerations

occupies about one-tenth of the book. It is brief and interesting. If anything, it receives more space than could be desired in this particular book, and the same can be said of the chapters on General Phenomena and Investigations, Surveys, etc. Without belittling these subjects, part of their space could well have been allotted to expanding the chapters dealing with Regulation Works proper. Or perhaps it should be stated thus: The book in question is worthy of praise: certain of its chapters rewritten and enlarged would find its readers with appetites stimulated by what the present volume offers. The chapter on Levees is especially valuable.

The book contains many references and several quotations. They are pertinent, indicate source or authority, and facilitate further reading on the subjects of the book. They are decided assets to the book. The book is used at the Engineer School of the United States Army in the course in River and Harbor Improvement. It is believed that graduate engineers interested in river improvement work will gladly give it a permanent place in their professional libraries.—G. R. L.

THE IMPROVEMENT OF RIVERS. A treatise on the methods employed for improving streams for open navigation, and for navigation by means of locks and dams. By B. F. Thomas, U. S. Principal Assistant Engineer, M. Am. Soc. C. E., and D. A. Watt, U. S. Assistant Engineer, M. Am. Soc. C. E. In two parts, of 333 and 380 pages of text, respectively (9 by 12 inches). Illustrated. Cloth. Second edition, rewritten and enlarged, 1913. John Wiley and Sons, New York. Price, $7.50 net.

This mention of the second edition of the well-known work of Thomas and Watt is not made through any recent discovery of such second edition, but largely to put it on record that PROFESSIONAL MEMOIRS is acquainted with it and has been since it was published, and to express regret that recognition has been so long withheld. The delay was in nowise due to insignificance or lack of merit in the new edition. And it is hoped that this tardy acknowledgment of a valuable contribution to river improvement literature will dispel any unfavorable impression that the slight that has seemingly been given it by this journal may have produced. Let it be stated at once that the new "Improvement of Rivers" is regarded as an excellent work. It has been adopted as one of the text-books in the River and Harbor Improvement course at the Engineer School, U. S. Army. It is deserving of the attention of all who are concerned, actively or otherwise, with river and canal works.

In view of the late date at which this is written, only a brief description of the book is here given. The majority of our readers are doubtless acquainted with its existence, and now have the book itself to refer to.

The new edition is in two parts, the former dealing with charac-

teristics of rivers; regulation; dredging and snagging; dikes; protection of banks; levees; storage reservoirs, and the improvement of the outlets of rivers. The second part has to do with improvement of rivers by canalization. The new edition contains over twice the number of pages of text of the former edition. It is illustrated by 84 plates and numerous halftones and cuts. Its examples and descriptions relate to foreign as well as to domestic river works and show careful and wide research on the part of its authors. Their labors have made easily available much valuable data which heretofore have been widely scattered. The arrangement of Part I can be criticised, but a good index minimizes its defect in this respect. And the proportionate space allotted to different subjects might be changed to the better satisfaction of some.

The book is entitled to much praise for the completeness of Part I, relating to canalization, especially for the section on movable dams; Part II would with its new matter and 38 plates have justified a new edition. The plates of both parts are admirable in their subjects and their quality.—G. R. L.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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MAJ. GEN. JAMES BIRDSEYE MCPHERSON

CADET, U. S. M. A., 1849-1852

BORN NOVEMBER 14, 1828; KILLED IN BATTLE JULY 22, 1864

(Portrait reproduced from "The Photographic History of the Civil War" by permission of the Review of Reviews Co.)

SEE PAGE 378

A Recent Trip to Italy, Egypt, Southern France and Spain.

*INTERESTING FACTS AND PERSONAL EXPERIENCES.**

BY

Dr. ELMER L. CORTHELL,
Civil and Consulting Engineer.

I will undertake to give you an outline and the principal features of a most interesting and instructive excursion from Bern to Bern, occupying three months of the winter of 1912-1913 and traversing Italy, Egypt, the Nile up to Assuan, the Mediterranean via Marseilles to Gibraltar, Southern Spain to Madrid, and thence to Bern via Paris.

As the visit contemplated not only historical scenes in Egypt and Spain, but also meeting engineering acquaintances and an examination of their important works, I may be excused for not going into detail upon the subjects treated in the guide books and which all tourists can see for themselves under competent guides, as I wish to draw attention to the engineering works of ancient and modern times and those now under construction.

At the International Congress of Navigation held at Philadelphia last summer, it was our duty and pleasure to receive and welcome the delegates and members from many countries on the docks, on their arrival at New York and Hoboken, on sixteen different steamships—about 250 delegates—arranging for their baggage and tickets for Philadelphia, or taking them to their hotels, and passing their baggage free without examination. This continued for ten or twelve days.

Then, after the Congress, were the extended excursions in the United States and Canada, where we did what we could to assist the delegates. And now you ask, "Why inflict us with these

*Contributed by Dr. Elmer L. Corthell, Civil and Consulting Engineer, 149 Broadway, New York, N. Y.

Congress matters?'' Because it was the intimate acquaintance, friendship and appreciation of these delegates that made our trip to Italy, Egypt, Spain and France much more interesting, instructive and enjoyable than it would otherwise have been, and furnished an ''open sesame'' to the very things we wished to see in all these countries.

In preparation for this voyage of three months, various books on the countries or parts of countries to be visited were sent for to Edward Stanford, London, and studied. Since our youth, Washington Irving's ''Tales of the Alhambra'' and J. S. C. Abbott's histories had never been forgotten; also the hotel list was carefully examined, and from the data obtained the points to be visited, and the time required at each, was decided upon. The best books were found to be Murray's ''Rome and Campagna,'' Murray's ''Egypt and the Sudan,'' Percy Wither's ''Egypt of Yesterday and To-Day,'' published by Grant Richards, London, 1909, and ''The Nile,'' by E. A. Wallis Budge, published by Thomas Cook & Son, and last, but not least, a small book, ''Egypt and the Nile,'' by Cook, which is absolutely necessary if the Nile voyage be made in a Cook's tourist steamer, and in Spain, Baedeker, and that lovely little ''Castilian Days,'' by the lamented John Hay. As Monsieur Voisin Bey, the aged (94 years old) constructor of the Suez Canal, had presented me two years before with his monumental work on the Suez Canal—seven volumes and a book of plates—which I had read carefully, this with a long-time knowledge of this great work, which till now I had never seen—all prepared us intelligently for the trip.

The question then arose as to the method of travel, and we decided to try the ''Inclusive Independent Travel'' system of Thomas Cook & Son of London, which, after our experience, we can heartily recommend; but let it be understood we hold no brief for this company nor for any of the hotels patronized. The latter were generally good, whether selected by us or by Cook. I wrote to the chief office at London, stating quite in detail what we wished to do and asked for an itinerary and estimate. For your information it should be stated that the system of ''Inclusive Independent Travel'' is a lump sum contract (a forfeit) by which by the payment in advance of the amount of the estimate Thos. Cook & Son undertake to take you through

and pay all expenses, including railroad and steamship and steamboat fares, hotels, conveyance to and from them, meals on trains, private guide and carriages for sightseeing, donkeys on the Nile voyage, gratuities to drivers and entrance fees to places of interest.

The detailed itinerary gives all information required, times of departure and arrival of trains; rooms are secured in advance by the London office; agencies in all the cities, or if no agencies exist, the hotels are notified to have a good guide and carriage at the hotel on a certain day and hour as given in the itinerary. On arrival at a station Cook's agent, or the hotel agent, is looking out for you. In fact, all expenses are included, and hotel, guide and meals in the train restaurants are paid by coupons bound in a small book, to be torn out in the order required. Expenses that are not included are excess baggage, station porters, hotel tips, laundry, etc., and you must not assume that these, and many others, are trifles.

The London office furnishes a conveniently bound pocket itinerary, about 6 by 4 inches (15 cm. by 10 cm.), which gives all details for each place on each page. The whole trip with stops took in the following places: Milan, Florence, Rome, Naples, Pompeii, Cava, Amalfi, Sorrento, Capri, Cairo, Ismailia, Nile voyage to Assuan, and return to Luxor and train to Cairo, Port Said, P. & O. steamer to Marseilles to Gibraltar, Algeciras, Granada, Seville, Cordova, Madrid, Toledo, Escorial, Paris.

I need not tell you that between Bern and Milan there is much to see of engineering interest and our personal acquaintance with Italian engineers has at various times helped much in examination of works; and let me say here that an acquaintance of many years with them and a study of their educational methods, ending with an examination of the Royal Technical School at Rome, close against that wonderful statue of Moses, of heroic size, led me to say in my report in 1898 on engineering education for the University of Chicago, that the thorough practical education of the Italian engineer fits him for his splendid constructions of the last six centuries, which we find all the way from the Italian lakes to Sicily, and gives the Italian that practical engineering sense that makes him a composite engineer, combining the technique of the Frenchman and the practical qualities of the Englishman.

A paper of great interest could be written on his irrigation and navigation works in the north of Italy—Lombardy and Piedmont—Milan and Turin. These works, some of them, at their initiation go back to the Thirteenth Century—650 years; and is it any wonder that with this long period and with extensive plains and with an innate genius for such work, we find all about us on the ride from the Alps to Milan and Turin perhaps the finest irrigation works in the world? The Naviglio Grande, an irrigation, navigation and power canal, utilizes the Ticino River coming down from the northern lakes and spreading its waters over an immense area by means of secondary canals and ditches. The Martesana Canal connects Milan with the Adda River on the East, and the Pavia Canal connects on the south with the Po and the Ticino. The Naviglio Grande is the oldest and the most interesting. It has been a gateway of commerce for over six hundred years. The Villoresi Canal, named from the engineer of that name, who worked hard for many years without avail to get his project adopted, but died before its realization, does not have its head gates at Lake Maggiore, as the Naviglio Grande does, but 15 miles below it, and it runs eastward, watering all the plain to the north of Milan. There is still another interesting canal to the south of Milan, the Vettabia, a private canal, owned and operated by a farmers' association. It is one of the most interesting in Italy. It was once a natural stream made up of three small creeks which came together at what is now the city of Milan, and the old canal or ditch of the year 1150 is now the "interior" canal of Milan and is known as the "Ring Canal," which is around the central part of the city.

The canals of Piedmont are equally interesting from a historical as well as engineering point of view. The principal canal is the Cavour, named from the great statesman who projected and urged it for many years.

About 1850, the government purchased quite a number of small canals, connected them into a comprehensive system and made them subsidiary to the larger project of a main canal from the Po, which, coming across the drainage of the country, would feed and equalize the smaller canals and ditches. This scheme was brought forward by Francesco Rossi of Vercelli and his plan was supported by Count Cavour, the best friend of irrigation Italy ever had, as well as one of the greatest statesmen

of modern times. It was a tremendous undertaking to dam up and divert a great river like the Po and carry it over one river here and under another river there. All the while Italy was struggling with Austria for its independence; its coffers were empty and the wars from 1859 to 1861 delayed the work. Cavour died in the latter year. In 1862, a syndicate of English and French capitalists took over the project, but financial troubles intervened and the company became bankrupt in 1867 and the project was tied up for three years by litigation. The government finally took over the whole work in 1874 after $10,000,000 had been spent, and it has operated it during the last thirty-eight years. There are many other canals in this section of the country. The whole of the north of Italy is intersected everywhere with canals and ditches, and the works of art are substantial—solid masonry—and are real works of art. (The above facts are based on a personal examination made in April, 1913.)

But we must hasten on to Rome, as all the intervening country is known to you and so, probably, is Rome. It was in Rome that we saw again several of our good friends of the Philadelphia Congress, whom we had welcomed on the docks at New York. They were all exceedingly kind to us. One day, by the entrée given to Commenditore Luiggi at the Victor Emmanuel Memorial now under construction and no doubt in size and grandeur surpassing any memorial monument in the world, we had the good fortune to go with him and the superintending architect to examine it from foundation to the lofty esplanade of the roof, and stand near the great bronze equestrian statue of Victor Emmanuel. I say "great statue" and you will appreciate its size, if you ever have the chance to ascend into it by way of the stairway in the hind leg of the horse and enter his interior, where 27 persons sat at table at a banquet the day of its inauguration.

An interesting engineering matter was brought to my attention by Mr. Luiggi, a recent project, or projects, suddenly brought out by the occupation of Tripoli by the Italian government after the Turkish-Italian war. As soon as the Italian government had secured a foothold there and was confident of holding its ground, it dispatched Commenditore Luiggi to examine the various harbor situations, make plans and set contractors at work. It was all started in an incredibly short time and these ports are now under construction. A map of Tripoli and the Red Sea coast will show you where they are. The plans not only contemplate

protection works, but modern ports and cities with water supply, sewerage, public buildings and all that is needed to make a port prosperous. Not only the port of Tripoli (Fig. 1), but two others on the northern coast are now under construction on Mr. Luiggi's plans and also a protected port on the Red Sea littoral at Massaua. (Fig. 2.)

We were introduced to one of the best known engineers of Italy, Mr. Italo Maganzini, who is the president (chairman) of one of the permanent commissions of government engineers. Mr. Maganzini has recently published an interesting report, or paper, on the subject of the engineering works on the large plain lying south of the River Po, called the Sub-Apennine and extending to the Adriatic at the mouth of the Po from Parma north of Bologna, including Ferrara and Codigoro near the sea. The pamphlet is well illustrated with photographs and hydro-electric works and by a detailed map showing the location of all the works and of the irrigation and navigation canals. (Fig. 3.)

I have said that Mr. Maganzini is chairman of a board. The government's works—railways, roads, and hydraulic works—are supervised by three boards of twelve superior engineers, with their office at Rome in the Ministry of Public Works. All propositions for concessions, all plans for works of all kinds that come before the Minister of Public Works are referred for examination and report to the appropriate board, which meets once a week to consider the subjects referred to it; and once each month, or oftener if called together, the three boards meet as a joint board to discuss and decide upon matters of interest common to all. In this way the work of the department proceeds under the advice of this very competent organization.

Our next visit was at Naples and that beautiful and interesting circuit of travel—Pompeii, where explorations are still going on bringing out treasures of the past; Cava, and that always delightful ride from there by way of Amalfi to lovely Sorrento and the sail to Capri. At Amalfi we visited, as all do, the Hotel des Capucines, where, since I was there in 1898, a slice of it consisting of eight rooms of this old monastery up on the hill, went down in a landslide, but the natural beauty of the place is not injured. The sun still shines bright and the same deep blue waters of the sea are beneath you, the same indented coast line, stretching away in the dim distance, dotted with villages on the steep slopes; and then there is that wonderful highway along the

bluffs toward Sorrento, with the Isles of the Sirens always in evidence. At Sorrento we visited the Crawford Villa, the proprietor no longer there and writing no more his beautiful stories, but the garden and the bold sheer bluff on the rock and the sea lapping its feet is as charming as ever. You will remember that Mrs. Harriet Beecher-Stowe, after a visit here and gathering the historical facts about the lovely girl of her story, wrote that

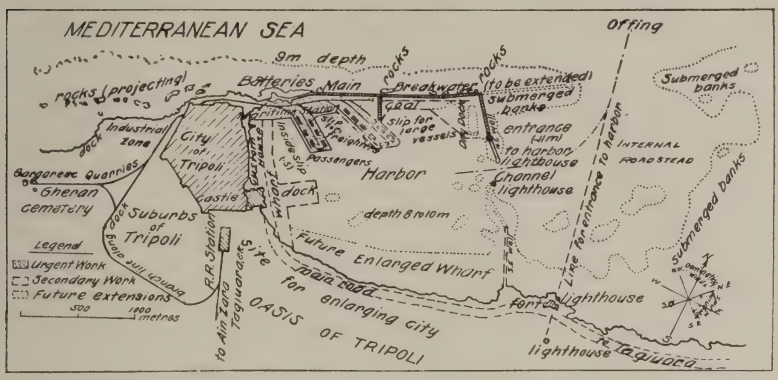


Fig. 1. General Map of Tripoli Harbor.

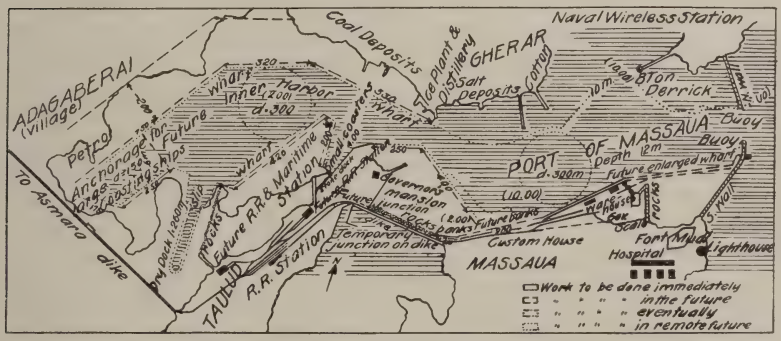


Fig. 2. General Map of Massaua Harbor.

classic, "Agnes of Sorrento," who lived in a house, or cabin, in one of the gorges about Sorrento.

Capri and the Blue Grotto you all know about, and I will not delay you there. We, then, on our return to Naples, sailed for Alexandria on the S. S. *Prinz Regent Luitpold*, of the North German Lloyd.

As we approach the harbor of Alexandria, the interest is enhanced by the facts of history, which all of us from our earliest

youth have treasured up with the abiding hope of some day seeing what we have read about, and possibly this lends a glamour to the land of Egypt, almost the cradle of the human race and a country contended for and possessed by nearly all the races of ancient times, as one after another gained predominance in Europe and western Asia.

We miss the Island of Pharos (as it is now connected with the main) with its lighthouse said to have been 600 feet (180 m.) high —one of the seven wonders of the world. It disappeared long since, destroyed by an earthquake about 1200 A. D., but on a clear, calm day its ruins are still visible in the deep water. It gave its name to all the light-houses since then—"Phare" in French.

The city founded by Alexander the Great in 332 B. C., containing in the First Century A. D. 300,000 population and adorned with palaces of kings and the arts of Greece and the wealth of Egypt and Rome, would scarcely be recognized by the ancient worthies, if they should visit it, for its then distinctive characteristics of royalty, literature and art have largely yielded to the demands of modern commerce, which has spent its energies and its money to prepare a deep and safe entrance from the sea for the largest modern steamers, and a quiet, convenient and well protected port with all of the latest facilities for handling passengers and cargo expeditiously from ship to rail and rail to ship and to send them forward on a first-class railway and equipment, at a good speed, to Cairo.

The entrance channel, until recently, had a depth of only 26 feet (7.9 m.); it now has over 30 feet (9 m.). A breakwater 1¾ miles (2.82 km.) long, extending from the southerly point of what was formerly the Island of Pharos, efficiently protects an area of 1,300 acres (525 hect.), of which 800 (325 hect.) have a depth ranging from 30 (9 m.) to 60 (18 m.) feet. Inside there is a solid mole for ships, 100 feet (30 m.) wide, constructed in 40 feet (12 m.) of water and 30 feet (9 m.) of mud, into which the foundation of rubble-stone was thrown. The mole is faced with large quarry blocks on the outside and with quay walls on the inside. With this mole 1,000 yards (900 m.) long and a great extension of walls around the port, there are about 2½ miles (4 km.) of stone quays, accommodating a large number of steamers and coastwise craft. Practically all the facilities of harbor and port—railway stations and railway to Cairo, were the work of English engineers and contractors.

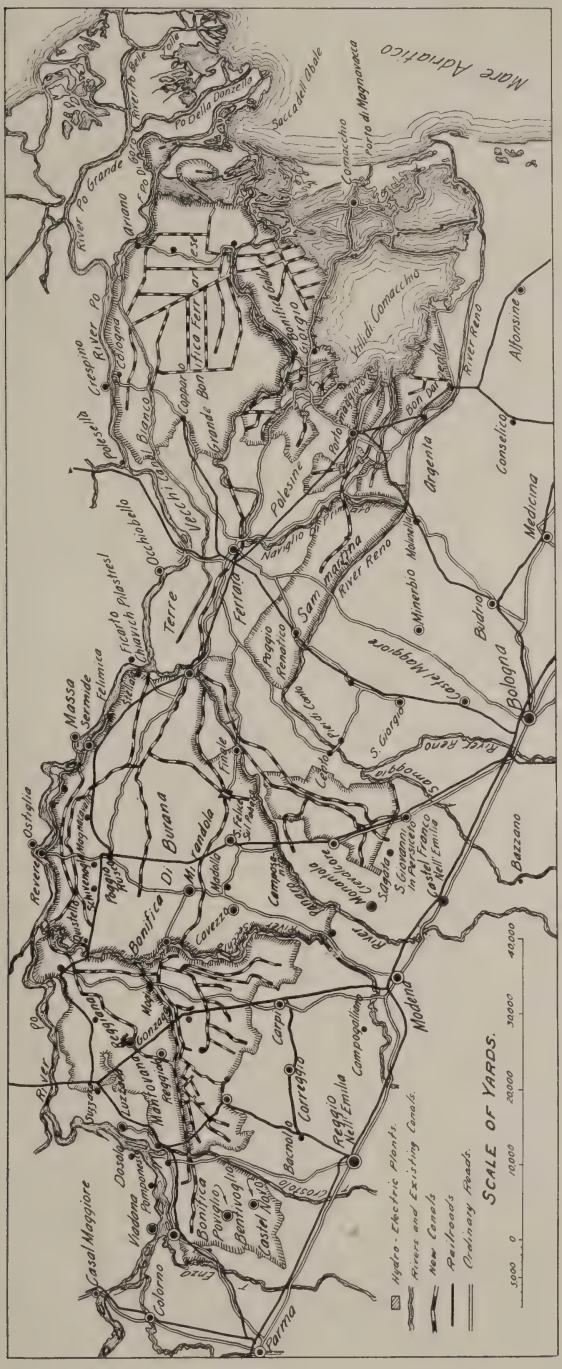


Fig. 3. Improvement of the Sub-Apennine Plain, from the Enza to the Sea.

As I had with me letters to English engineers in Egypt, members of the Institution C. E., sent me as a member of the Institution by Doctor Tudsbery, the Secretary, I was quite sure that my hopes of seeing engineering works would be realized, but I was not quite prepared for the generous attention given me by them, particularly by Mr. C. Carkeet James, Controller General of the Cairo Main Drainage, and Mr. Murdoch Macdonald, Under Secretary of Public Works. As we knew personally Mr. Perrier, Chief Engineer of the Suez Canal, who attended, as delegate of his Company, the International Navigation Congress at Philadelphia, we were quite confident he would do all in his power to show us his important work, and our expectations were more than realized. I should add that Major Tucker, Royal Engineer, engaged in building reinforced concrete barracks for a force of 5,000 soldiers—infantry, artillery, cavalry, and a *camel* brigade—was kind enough to take us over his work not far from Heliopolis, in the suburbs of Cairo, and show us one of the most complete layouts—substantial, convenient, comfortable and admirably arranged, built on the sands of the desert, with well paved streets, walks and gardens, and grass plots and shrubbery started, all well watered and sewered. It is well worth a visit by any one stopping a few days at Cairo. There is much of engineering interest in this city of three quarters of a million, and rapidly growing.

There are three bridges across the Nile. The "Roda Bridge" was built by Sir William Arrol Co.; it is 1,750 feet (535 m.) long, with spans of 140 feet (42 m.). The piers were sunk to a depth of 70 feet (21 m.) below low Nile level, or 92 feet (28 m.) below high Nile. The bridge was built in twelve working months —a good record. A fourth bridge, the "Abbas Bridge," named after the reigning Khedive, is planned on the pattern of the Tower Bridge of London, in order to avoid the interference and inconvenience caused by the existing swing bridges. There has recently been completed, and opened to traffic, the Boulac and Zamalek Bridge between Cairo and Gezira. This bridge was built under difficult conditions, on account of the very considerable depth of water. It was impracticable to build a staging from which to lower the caissons to the river bed or for the erection of the spans. The caissons were hung to powerful jacks supported on two pontons, anchored over the site of the piers

and held there until the caissons were well sunk and stable in the river bed. The spans were built on a staging on shore and floated out to position on two pontons.

In the official inauguration of this bridge by Sirry Pasha, Minister of Public Works, he referred to the rapid growth and extension of Cairo in recent years. Four years ago the Khedive opened the Roda Bridge, connecting Old Cairo with Gizeh and the Pyramids. Last summer a new highway was opened between Cairo and the baths of Heluan, about 15 miles (24 km.) by a good highway. The island of Gezira, which a few years ago was a rural district, has now a large suburban population. "Old Cairo" is rapidly disappearing and the miserable huts are being removed, the unsanitary situation greatly improved and space given for a new city there.

One of the most important works is that in the hands of Mr. James, which is so novel in its conditions, plans and general situation that it merits more than a brief notice. I may say in passing, that when we went to Assuan by the *Egypt*, one of the latest and largest tourist steamers of Thos. Cook & Son, on a voyage of nearly three weeks, Mr. James, to whom I had presented my letter of introduction, showed me some of his plans and insisted that we should give him a day on our return, to make a personal inspection of the works, which were then in the full swing of construction. We therefore shortened our Nile journey two days, returning to Cairo by night train from Luxor, instead of from Assiut, as we had arranged. This gave us one day for him and one for Major Tucker and then time to get to our steamer, *China,* of the Peninsular and Oriental Co., at Port Said for Marseilles and Gibraltar.

It was a beautiful day for the main drainage examination. Mr. James is one of those engineers who have all details at command, but he has also a corps of able assistants in immediate charge of each work, all of which, however, is under contract, and the engineers of the contractors were on the job, as they well might be, for some of them were struggling with trenches 30 feet (9 m.) deep in running quick-sands; for it must be said here that the subsurface water of the Nile extends to the highland on each side through the alluvial sands, and (a most interesting feature) has a varying slope. As the waters of the river rise, the slope towards the highland steepens and when the river falls, the level near the highlands still continues to rise until at last the slope is

reversed and then the subsurface water next the high lands begins to fall and continues to fall even after the river rises, and then it rises again. The valley at Cairo is about 15 miles (24 km.) wide and yet this movement of subsurface water is always in evidence in wells dug anywhere the entire width of the valley. This phenomenon exists everywhere, even in the delta itself, the head of which is about at Cairo. The length of the delta to the sea is about 90 miles (145 km.) and the width there about 90 miles (145 km.) also. It is therefore a true delta △, and yet the entire area is underlain with the percolating water of the Nile, about which we will have something interesting to say later on.

The "Site Plan of the Cairo Main Drainage Works," which accompanies this description (Fig. 4), gives the outline of the project and of the territory traversed, and of the sewage farm of 3,000 acres in the distant desert, about 16 miles from the center of Cairo, and the "legend" on the plan shows the peculiarities of the plans to meet the rather unique conditions and to dispose effectively of the sewage of a population of now over three quarters of a million, most of whom have not been accustomed to modern sanitation. It has been a question with some, whether this rather ignorant people, as many are, can be persuaded to use the modern facilities that are to be forced upon them; but every house, however old and humble, will have plenty of water and the most modern and convenient sanitary arrangements, which will in course of time change the habits of centuries.

Apart from the engineering interest in this locality, there are very interesting historical features which the tourist goes into the desert to examine, and who, travelling along with his Baedeker, no doubt wonders what the purpose is of this deep trench nearby his desert route, where hundreds of Egyptian laborers and batteries of boilers and pumps are hard at work.

As we went along in our comfortable automobiles and train and trolleys—for there is a good highway or track built by the Main Drainage Works the entire length of the line, after leaving the railroad, to the sewage farm—we passed the new barracks of Major Tucker on the right and those of the Egyptian army, the Kubba Palace of the Khedive, with its beautiful grounds, stables, and farms, built by Ismail Pasha for his son Tewfik, and where the Khedive often resides; we pass, about 6 miles (9.7 km.) out,

Mataria, formerly renowned for its balsam plants brought from Judea by Cleopatra, and near by is the new city of the old Greek city Heliopolis, and in the Jewish scriptures "On," which is to be one of the most beautiful suburbs of any city, and where millions are being spent to make it so. Already it has one of the

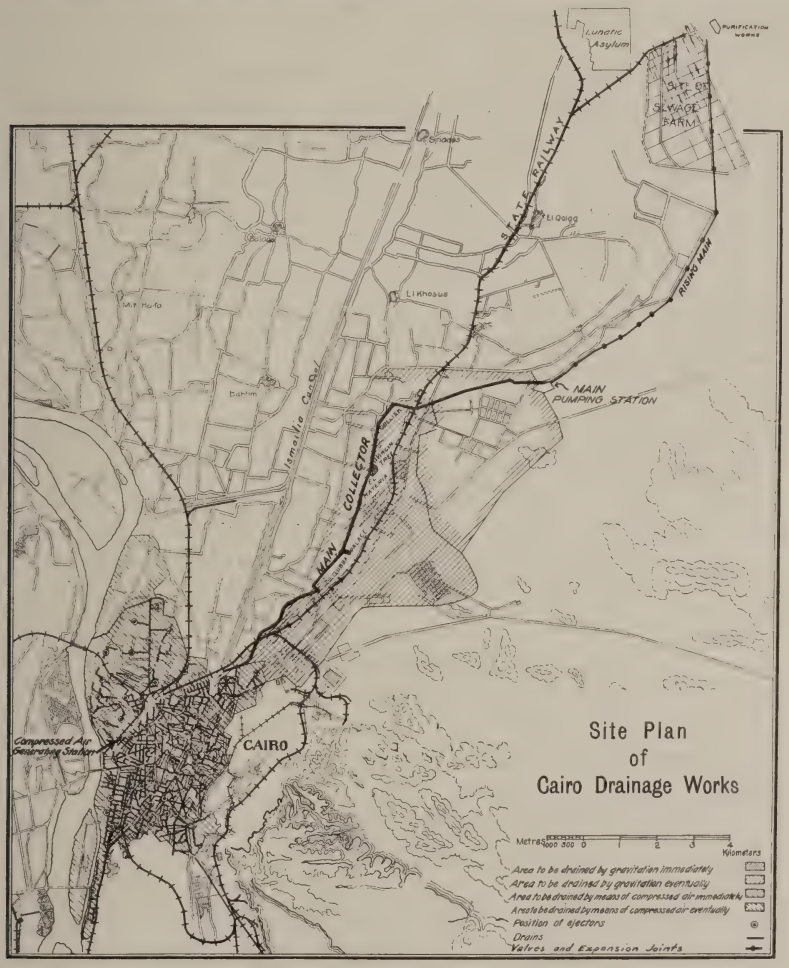


Fig. 4.

largest and finest hotels, and hundreds of artistic and convenient residences, the project of a syndicate, mostly Belgian. Near by is the "Virgin Tree," where the Holy Family stopped on their flight to Egypt. Here is the old sycamore under which, or one of its predecessors, they were shielded from the heat of the noon-

day sun. These old trees have succeeded each other ever since; the one now just alive and propped up was planted in *1672*, the former one having died in 1665, and to replace the present one a shoot of it was set out near by and that, too, will in the year 2100, no doubt, be replaced in the same way, so as to mark forever this sacred spot and the old well near by, where Joseph and Mary drank.

We now pass, and so does the Main Collector, near the lone Obelisk, 66 feet (20 m.) high, the oldest in Egypt, built by Usertsen I, "The Son of the Sun God Ra," about the year *2433* B. C. "Has executed this work at the beginning of a thirty years' cycle, He to whom life may be given for evermore," so the inscriptions cut into the Obelisk say, and much more. Its fellows, of which there were several, have been removed from time to time; one to Rome, two to Alexandria, and then one of these to the Thames Embankment, London, and the other to Central Park, New York; so it stands here near Heliopolis the only remaining vestige of Usertsen I, and may he "the son of the Sun God Ra," forgive a sewer with the sewage of his descendents 4,346 years afterwards, going for purification and the fertilization of the desert, miles farther yet, to raise shade trees and garden sauce for a great city. We are not far from another interesting spot, where the "Mounds of the Jews" locate the ancient city of "Onion." From the top is a beautiful view, and the same can be seen from the top of the house of the ostrich farm, which we visited another day, not far from Heliopolis. To the south the Pyramids (Cheops and the others) and Cairo with the Citadel, standing out prominently in the clear cloudless sky, and nearer to us is the lone Obelisk, the palace of the Khedive, and far to the east the range of high hills against the sky. The Jewish historian Josephus says that the son of Onias, the High Priest, fled from Antiochus, the King of Syria, and took refuge in Alexandria in the time of Ptolemy Philometer, about 180 B. C., and got permission from him to build a "temple" like that at Jerusalem," led to do so by a prophecy of the Prophet Isaiah, who predicted that there would be a temple in Egypt built by a Jew. Other Jewish cities were built near here afterwards: They were probably, according to Isaiah, the "five cities of the land of Egypt," which were to "speak the language of Canaan," and now a sewage farm is located among all these relics of a great past and the sewer that brings the

sewage lies beneath and alongside of the remains of the edifices of the ancient Egyptians, Phœnicians, Persians, Greeks, Romans, Jews and Christians, and over these desert plains armies have fought and contended for supremacy, and the hordes of Asia, coming from their far distant countries via the Isthmus of Suez, have swarmed over the country and conquered and were in turn conquered.

The history of this interesting country is so enticing to us that modern works seem almost out of place, but when we know that they are for the great benefit of a populous country, we must forget the past and take up with the always interesting present.

The Site Plan and its notes and the splendid portfolio of detail contract drawings show this interesting work of the main drainage clearer than any written description. It will be seen that a complete system of ejector areas covers the entire city and is to be extended in the future. These ejectors are operated by compressed air from a main plant, on the western edge of the city. These all work automatically as they fill and eject the sewage into the smaller sewers, all of which discharge into larger collectors and these into the "Main Collector," which extends by gravity about 9 miles from the city to the main pumping station, which pumps the sewage to the purification works at the sewage farm, about 7 miles farther out by a "Rising Main."

The Main Collector is circular, 1.6 m. (5.2 feet) in diameter in a mass of concrete with a circular concrete drain below it 1 foot in diameter, laid in and surrounded by rubble-stone.

There are forty-three working plans in all,* which are illustrative of the entire work.

The works, as Mr. James says, are not complete, but they are everywhere under construction. At Khanka (at the sewage farm) the process of raising shade trees in a large nursery is in operation by water from a deep well, operated at present by a large windmill, and it is remarkable how quickly the desert responds to the application of water, just like the deserts of our Far West. They need only water to "blossom like the rose." No doubt, the sewage farm will supply the city of Cairo with most of its garden products.

As I have already said, I have had great interest in the Suez Canal for many years. I find in my "Bibliography of 63 papers

*Not printed.

and books published'' No. 16, as follows: ''Ship Canals, Johnson's Universal Cyclopedia, Vol. XII, 1886.'' One of the Ship Canals quite fully described was the *Suez* Canal, compiled from reports and books on the subject, principally Professor Nourse's history, and from considerable correspondence. My intimate connection with the American Isthmian question and many years of study and much writing of reports on maritime commerce since 1886, and particularly in the last two years, investigating and presenting a ''Report'' upon the ''Question'' before the Philadelphia Navigation Congress: ''Dimensions to be given to Maritime Canals, taking into account the probable dimensions of ocean ships of the future,'' and in which I discussed the increasing depth and dimensions of the Suez Canal—all this naturally gave me a strong desire to examine it under the direction of its chief engineer. The results of this examination will be given a little later.

Reviewing a little of what I wrote many years ago, let me say that like the American Isthmian projects, efforts partially successful characterize all the past, and the hopes of commerce have centered upon the two isthmian problems, and both will soon be solved. The ancient attempts at Suez confined themselves to a connection between the Red Sea and the Nile, and what was left of one of them is now the ''Fresh or Sweet Water Canal,'' from the Nile to Ismailia and thence to Suez and Port Said, which gives the existing main salt water canal its sweet water for domestic and potable purposes, supplying the three principal cities and all intermediate points. It is appropriate here to remark that at each of the three cities there is an adequate plant, filtering basins, pumps and reservoirs, and water as pure as any in the world. At each installation we examined the water, and at Port Said, where the service, which furnishes not only the city and the canal works but also passing steamers, is in charge of a superintendent who is a chemist, the record shows that water delivered to consumers contains not more than *6 microbes per cubic meter* (35 cubic feet). If more appears by the analysis, that basin is drawn down and cleared out. The water in the first basin (there are eight in all), as it comes from the canal, contains less than 125 microbes per meter, and yet on the main Nile, where there is an immense volume of running water under the purifying influence of sun and air and

wind, we are told to drink special table water, because Nile water is unhealthy.

But to go back into the past history of the canal. On the north wall at Kanak, near Luxor, is a scene of the time of Seto I, 1366 B. C., showing his triumphant return from Asia by way of "The Cutting," which probably communicated with the Nile, as crocodiles are seen in the hieroglyphics. Necho II, 612 B. C., who defeated Josiah, King of Judah, built a canal which tapped the Nile at Bubastis (the Pibeseth of the Bible) on the west bank of the Pelusiac branch of the Nile, the mouth of which is *east* of the present ship canal. Necho's canal reached to near the present town of Zagazig and follows almost the line of the present Wadi Canal to Heliopolis. In those days, the Red Sea extended much farther inland than now; also the delta of the Nile since then has stretched farther seaward into the Mediterranean, so that it is not easy to locate the work of these ancient worthies. Darius, one hundred years after Necho, cleaned out this canal, badly silted up, and made it navigable for the boats of those days, so that the passage between the two seas was made by sailing up the Pelusiac branch of the Nile to Bubastis and along the canal to Heliopolis, where the cargo was transshipped to Red Sea vessels. The next Egyptian king, Ptolemy Philadelphus, about 285 B. C., again cleaned out the canals and put locks in at Heliopolis, but which prevented the salt water from mingling with and spoiling the fresh water. But, as the Nile thereafter almost entirely deserted the Pelusiac branch, this route passed into a state of desuetude, for, in the reign of Cleopatra, 31 B. C., it was impassable. There is not much more to say for this route, though it is said that in the year 1000 A. D. it was opened up again, but after that it was choked up with sand and neglected. In 1571 the Venetians, who then had a monopoly of the trade with the east, began negotiations with the Sultan of Egypt, but they were stopped by the Turkish invasion of Egypt, after which all idea of a canal was given up. In more modern times Napoleon Bonaparte undertook the investigation of an interoceanic canal, and he himself examined the old canal of Necho and his successors and ordered Monsieur Lepère to make a survey. He reported a difference of level of *30 feet* (9 m.) between the Red Sea and the Mediterranean, which, of course, would make a maritime canal without locks impossible. In 1846 a mixed com-

mission, including George Stephenson, was appointed to examine into the practicability of a canal. They exploded Monsieur Lepère's old error, but they came to no conclusion. Then came on the scene Ferdinand de Lesseps. A project was drawn up under his supervision by Messrs. Linant Bey and Mougel Bey, under a concession from the Viceroy of Egypt, Said Pasha. This project was submitted to an international commission in 1856, comprising representatives from Austria, England, France, Italy, the Netherlands, Prussia and Spain. The project, modified by this commission, was accepted and successfully carried out, as we all now know.

It is unnecessary to give its history. In my "Report" to the Philadelphia (1912) Navigation Congress (alluded to above), I traced the successive enlargements in the project of a ship canal on the American Isthmus, due to the increasing size of vessels. This happened on all the routes proposed and I had the following to say about the Suez Canal, which is apropos at this point in this communication:

The Suez Canal has also had many vicissitudes and a constant development due to the commercial requirements. The steamships have been obliged, greatly to the detriment of trade, on the long course from Western Europe to the Orient, to keep their dimensions at a minimum—especially their draught—but the insistence of commerce has continually forced the canal company to make greater dimensions.

No doubt the competition of the Panama Canal, with its much larger dimensions, will compel the Suez Canal Company to give to commerce what it requires—so that competition will redound to the benefit of the whole world.

It may not be generally known that as long ago as 1855-1856 an International Commission reported that the depth should be 8 m. (26.2 feet) and the width 64 m. (210 feet) at the bottom and 97.5 m. (320 feet) at the surface of the water for two lines of vessels in movement and a third line tied up at the bank, between the Red Sea and Bitter Lakes; and between there and Port Said 43.9 m. (144 feet) at bottom and 79.8 m. (262 feet) at top, but in 1862 these, for that period generous, dimensions, were, on account of finances, unfortunately reduced to a width of 22 m. (72 feet) bottom width, much to the detriment of the world's commerce for many years.

The Canal was opened to traffic in 1869 with these dimensions —but there was a continual demand for greater depth. In 1884-1885, the International Commission recommended a *final* deepening to 9 m. (29½ feet) with 65 m. (213 feet) width from

Port Said to the Bitter Lakes and 75 m. (246 feet) from there to the Red Sea.

In 1899 this "final" depth of 9 m. was reached, but then commerce still needed more, and it was decided to make it 9.50 m. (31 feet). Then the consensus of opinion demanded 10 m. (32.8 feet), and still there was not sufficient depth, and now the final (?) depth is to be 10.50 m. (34.5 feet).

Regardless of the fact that the canal—in spite of its physical restrictions upon commerce—was earning enormous dividends, it has required constant urging to induce the company to hasten its work and give trade a freer passage.

The following, stated by Sir John Wolfe Barry at the Engineering Conference, London, in 1907, confirms the above statement: "I happen to have been on the technical Commission of the Suez Canal for many years with my friend Sir Charles Hartley and we—the British representative engineers—have persistently, year after year, in season and out of season, for the last fifteen years, urged upon the authorities of the Suez Canal the extreme importance of deepening the canal to the depth which is now nearly realized, of 10 meters."

Evidently the English engineers on the Commission can not be criticised for the delay in making the channel more adequate, and from our knowledge of the French engineers, the same may be confidently said of them.

In the author's opinion neither 10 nor 10.50 nor 11 meters will suffice. He believes that within a few years this canal will necessarily have to be deepened, in order to retain its traffic, to 13 meters (42.6 feet), at least—allowing for a draught of 12 meters (39.4 feet) with corresponding width to allow for the passage, in two lines, of vessels of 25,000 tons.

Note the gradual and evidently reluctant concessions to the steamships in the matter of draught. The company authorized in 1869, 7.5 m. (24.6 feet) draught; 1890, 7.80 m. (25.6 feet); 1898, it widened the canal from 22 m. (72 feet) to 37 m. (121 feet); 1902, 8 m. (26.2 feet) draught was authorized; 1906, 8.23 m. (27 feet); 1908, 8.53 m. (28 feet), the canal having been deepened to 9.50 m. (31.2 feet), and 8.53 m. (28 feet) was the authorized draught in January, 1911.

With the above in mind let us see what we found by our personal examination. From the profile and sections handed to me by Mr. Perrier, Chief Engineer, after finishing the examination of the Canal on January 25th last and the new programme of improvements of the works decided upon at the reunion of the International Commission in 1912, it is seen that provision is made for a depth of *12 meters* (practically 40 feet) everywhere, with a minimum or standard width at the bottom of 60 m. (197

feet). As it has been found necessary to allow a depth of 1 meter under the vessel when moving, the draught can then be 11 meters (36 feet). Comparing this uniform depth of 12 meters with depths at Panama, we find at the latter canal the following as the final depth: Limon Bay to Gatun Locks 41 feet (12.5 m.) at mean tide, range of tide 1½ feet (46 cm.) or at mean low tide 40.3 feet (12.3 m.) In Culebra Cut 45 feet (13.7 m.), over the lock sills *40* feet (12 m.). The tidal fluctuations at the Suez Canal are practically nil on the Mediterranean side, depending upon the winds. On the Gulf of Suez, and also depending largely upon the winds, the difference may be assumed at about 3 feet (0.9 m.), so that at times the depth will be 38½ feet (11.7 m.) but generally the difference of levels is inconsiderable, so that the two canals will have about the same usable depth.

I am very glad indeed to say that the able International Commission, all of whom are competent to appreciate the requirements of commerce, have not only decided upon 12 meters (40 feet) depth, but have been able to persuade the canal company to accept their conclusions. Twelve meters is a great deal better than the 10.50 meters stated by me to be "final" in March, 1911, when my "Report" to the Philadelphia (1912) Navigation Congress was finished. There is no question in my mind of the ability and policy of the International Commission, and the company as well, to keep abreast of the times and up to the requirements of commerce. Not only is the depth increasing but the alignment is being improved, the under-water slopes in many places are being flattened and the foot of the slopes at the water line is being protected (see Fig. 5), and at Port Said a radical change in and enlargement of facilities is now being carried out at a cost of 15,000,000 francs ($3,000,000). All the facilities, including the fleet of dredges, tugs, boats, etc., of the company and all of the repair shops will be concentrated on the east side on an area of 800 meters (2,624 feet) wide and 2,500 meters (8,200 feet) long, on ground reclaimed from the sea by means of a substantial sea wall and filled behind with sand, pumped over by hydraulic dredges. There will be dry docks, slipways and smaller cradles, a complete electrical equipment of three 600 H. P. motors, shops of all kinds, storehouses and offices, all substantial structures of brick, stone, concrete and reinforced concrete, and an abundant supply of fresh water by means of submarine pipes from the water works on the west side.

Some of my pencil notes taken on the examination of the canal are of interest. The company has one floating crane of 60 tons, one of 40 and one of 35 tons. It has at work twenty dredges of various kinds and two Lobnitz rock breakers, the breakers weighing 20 tons. The dredging contractors have four dredges. Widening according to the programme is going on in many places. The canal and sea approaches are well lighted by Pintsch gas

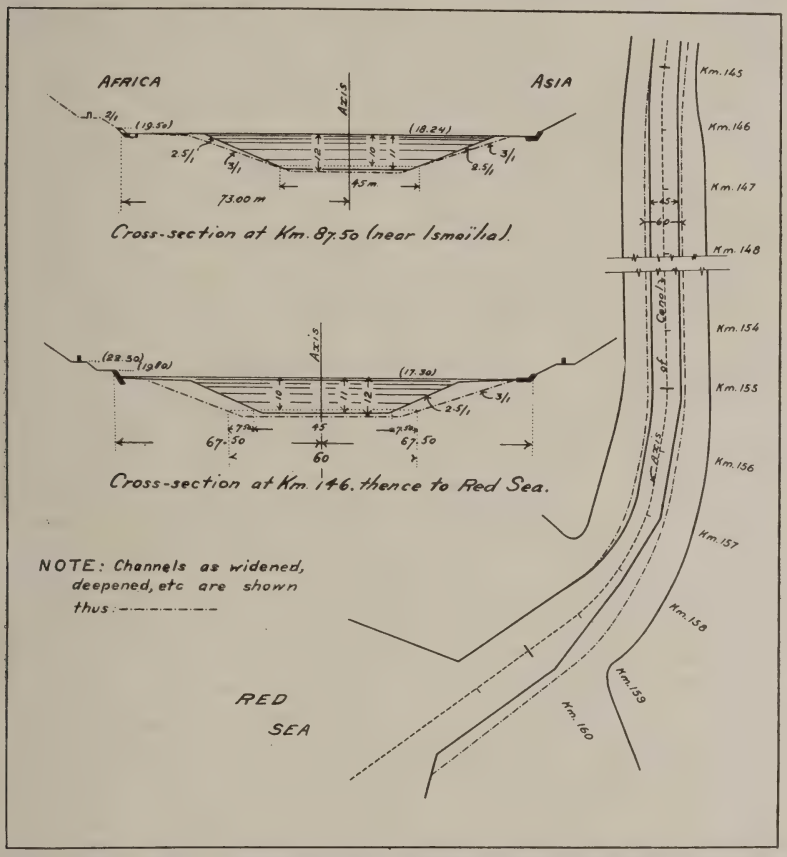


Fig. 5. Suez Canal. Typical Plan and Sections showing widening, deepening, etc., under way.

buoys. There are many day and lighted beacons on land and on pile platforms at the wide places and in the Bitter Lakes where necessary. The least radius of curvature now is 2,500 m. (8,200 feet). There are fifteen observation stations along the canal, provided with telegraph and telephone. Every ship that passes through is reported to the principal office at Ismailia as it passes

the observation stations, and on a diagram at the main office the name of the steamer and of its commander and its tonnage are plotted, as well as the time of any stoppage. This diagram looks very much like the train sheet of a railroad or the Marconi sheet on shipboard, showing steamers going in reverse directions. By this means every incident of the passage is known at the head office. The allowable speed is 10 km. (6.2 miles). The average annual cost of maintenance is 5,000,000 francs ($1,000,000) and of construction work 6,000,000 francs ($1,200,000).

It is unnecessary to state that the jetties at Port Said have been extended and that they protect the entrance against the storm waves and that the approach at the Suez end, where scattered shoals exist, is well lighted by a light-house and numerous lighted buoys. The width of the two channels between these lighted shoals, the inner and outer, are 300 m. (984 feet) and 375 m. (1,230 feet), respectively.

The income and traffic of the canal are constantly increasing. During the last year, regardless of lowering the rates (now 6 francs or $1.20), which cost the company 10,000,000 francs ($2,000,000), there has been a notable increase in revenue and tonnage, and no doubt the enlargement of the canal now going on, permitting the transit of steamers of much larger tonnage, will greatly augment the paying tonnage and lead to further reductions in canal dues. The income in 1912 was 135,720,000 francs ($27,144,000), an increase of 1,710,000 francs ($342,000) over that of 1911. The net register tonnage was 20,275,120 tons, being 1,950,326 tons over that of 1911.

In closing the Suez Canal part of this address, I wish to say that I found nothing to criticize, everything to commend, and I am glad to retract what may have seemed to be a complaint of indifference in my paper presented to the Navigation Congress at Philadelphia in 1912. "Familiarity has not bred contempt," but esteem and confidence in the pronounced policy of enlargement of the facilities for the commerce of the world.

THE NILE.

After all, *it* is the "Mecca" of all travellers. Cairo, comparatively modern, is interesting. Its sanitary works, already described, arrest the attention of an engineer; the Suez Canal also; but when we get on the Nile we are in Egypt of old and we do not wonder that the Nile was worshipped or that it was the

maker and benefactor of all the races that have inhabited its valley and lived by its waters. It certainly is unique among all the rivers of the world, in that it never ceases its flow through the desert sands, though for a thousand miles no rain nourishes it, and this gives to it something mysterious, and to its inhabitants it is something sacred. As we travel upon it and daily visit the temples and tombs of departed kings, priests and great men, see their methods of building, begin to understand why the great built their tombs in the living rock, generally far away from the alluvial lands, and gaze into their sarcophagi and sometimes upon the king or priest himself, and go down into the tomb—as of Seti I, passage after passage, room after room, until finally where Belzoni in 1817 found him—and other tombs 300 feet down into the rock; and then look at the great pyramid of Cheops near Cairo and wonder why all this costly work was done, we learn that their belief that there were two natures in every man, one the body we live in, the other his "Ka" or double, who visits the body after death, led them to be extremely careful about this body. The central idea was to hide it effectually from any possible disturbance and to supply it with suitable offerings, and, as the "Ka" could not visit the places where it lived on earth, there must be provided pictures on the walls of the tomb that would show it what it did and had and saw in its earthly life, so that it would feel "at home" when it came to visit the preserved body; and when we see in the Cairo museum the body of Seti I and especially of Rameses II (The Great) and see his uplifted arm, as if giving his royal command, we begin to respect the motives of these great men that could and did do such remarkable things in their lives, and to cease from our smiles at their singular beliefs about the future life.

The voyage up the Nile in one of the comfortable tourist steamers, where all the conveniences of the best ocean steamers are ours, with the daily visits on shore, the exhilarating donkey rides in the clear, cool air of the morning, the examinations of temples and tombs and the companionship of your fellow passengers and the perennial sunshine, then the return to the boat, appetites whetted for a good luncheon, which you will certainly get, the bartering with the natives who line the bank at every town with all sorts of native products; then the sail along the banks, watching the "shadoofs," as the half-naked Egyptian fellaheen work the well-sweeps, raising the Nile water by some-

times five successive sweeps up perhaps 20 feet from the water into the surface ditches of their fields; meeting the "feluccas" (sailing boats) and the steam and sailing dahabeahs, finding everywhere on the trip of nearly 600 miles (966 km.) from Cairo to Assuan always something new and interesting, and wherever we are, on board or on shore or in temple or tomb, we know that Rachid, the "All knowing" principal dragoman, can answer all our questions and can tell us things of the greatest interest that we can not find in the guide books—all these things make it worth while to spend several weeks on the Nile.

These principal dragomans are wonderful men, know all the languages, have travelled nearly everywhere in the world, read and study between tourist seasons and keep abreast of all new discoveries. They are gentle, courteous, clear in statements, treating all respectfully, listening to all questions, however silly, and dispensing their wealth of information prodigally and never wearying of our importunities. The native, be he Mussulman or Christian, Arab or Copt, is intelligent and trustworthy and even the "donkey boys" who escort us on the excursions are kind and faithful fellows, and we can but admire the trim, supple, soldierly police of the Government Camel Brigade who, one, sometimes two, accompany every party on the excursions to see that the crowd of donkey boys and the people of the villages do not molest or annoy us and to keep all in order.

But all this you can read about anywhere. I am more desirous of telling you something of the Nile and its hydraulic and irrigation features and give you the information obtained at first hand from the English engineers whom I met, and who gave me freely all I asked for. Let me here tell you who they are, as you may meet them if you ever make a trip to Egypt—Mr. Murdoch Macdonald, M. Inst. C. E., C. M. G., now Under Secretary of Public Works, formerly the engineer of the Nile reservoir at Assuan, but now in the Department at Cairo and in charge, with other matters, of the important delta drainage works which I will tell you about a little later; Mr. David A. Watt, engineer in charge of the Nile reservoir; Mr. E. Lambton, engineer at the principal administration office at Assiut, and Mr. McClure, the engineer of Messrs. John Aird Co., contractors, and who had charge of nearly all the work done by them, which com-

prised the Assuan, Assiut, and Esneh barrages. Briefly, now, the main hydraulic features of the river.

The sources are Lake Victoria Nyanza (which means "lake"), Lake Albert Edward and the Abyssinian Mountains. Lake Victoria, the farthest south, is near the equator in the region of perpetual rains, the longest and most southerly tributary of the lake being the Kagera River. Counting in the lake and this river, the total length of the Nile is about 4,000 miles (6,440 km.), which probably makes it the longest river in the world, although its maximum discharge can not compare with those of the Mississippi or the Amazon; all of which are respectively and approximately 500,000, 2,000,000 and 5,000,000 second-feet. Strictly speaking, the Nile is formed by the junction at Khartum of the White Nile coming from the lakes and the Blue Nile from Abyssinia, and still farther north the Atbara. Below this point there are practically no more tributaries in its course of 1,600 miles (2575 km.) to the sea, which makes the river phenomenal. As to altitudes, Lake Victoria, 3,700 miles (5955 km.) from the sea and in area 25,000 square miles (65,000 km.^{2})—larger than any of the Great Lakes of North America, Superior excepted; Huron has 23,200—is 3,672 feet (1,120 m.) above sea level. Lake Albert is 3,350 miles (5,390 km.) from the sea and its elevation is 2,250 feet (686 m.). At about 2,700 miles (4,345 km.) from the sea the river runs at a very small slope and it is here that is found the great obstruction of the "Sudd," a vegetable growth often very extensive, and the river becomes a swamp. One is reported as three-fourths of a mile long, perfectly firm and overgrown with high reeds and grass. The graves of people who had died of the plague were actually upon it. The Nile plunges under them by a subterranean channel, roaring like a cataract. These obstructions receive accretions of other floating islands which drift down to them. There have been found upon them trees and dead crocodiles and hippotami. This "sudd" is met with over a stretch of 250 miles (403 km.). The English government has endeavored at considerable expense to make a channel through the various sudds. A party of 700 men and four officers and four steamers in 1899 were sent up to remove nine blocks of sudd. It was temporarily removed then and a channel opened. Recently systematic attempts have been made by dredging, and Mr. A. W. Robinson, of Montreal, has two of his dredges at work there.

The most important characteristic of the Nile, outside of its long course without tributaries or rain, is the alluvium which it brings down to fertilize the desert lands of Egypt. The White Nile brings down a fine, whitish clay, hence its name. But the Blue Nile and the Atbara are really the builders of Egypt. They bring down from the Abyssinian Mountains the rich mud, volcanic dust, which enriches annually the millions of acres by the inundations. It has been well said that "Egypt is the gift of the Blue Nile and the Atbara." The Blue Nile is 960 miles (1,546 km.) long, and the Atbara 700 miles (1,127 km.).

The ancient Egyptians kept a careful record of the height of the Nile flood by means of "Nilometers" at Philæ, just above the Assuan dam and now in the reservoir formed by it, on Elephantine Island, opposite the town of Assuan, below the rapids at Kanak, at Edfu, Esneh, and on the Island of Rhoda at Cairo. These generally comprised a well in which was a pillar marked off into cubits, and at the bottom an opening into the Nile. The pillar at Rhoda was divided into 17 cubits of about 21½ inches (55 cm.) each; the 10 upper ones divided into 24 digits each. The walls of the building are decorated with inscriptions, and there are also passages from the Koran relating to the "water sent by God from Heaven." When this Nilometer was constructed, a reading of 16 cubits meant the level at which flood irrigation could be assured everywhere. The level now is 20½ cubits, the difference showing the rise of the river bed, which amounts to about 5 inches (13 cm.) in one hundred years, or about 1/16 inch (1.6 mm.) per annum. From measurements at the Elephantine Nilometer the rise is about 3 meters, say 10 feet above the bed 1900 years ago, or about 6/100 inch (1.5 mm.) per annum. This Nilometer is very interesting. Plutarch says that the Nile rose on it to the height of 28 cubits; and at Edfu there is an interesting text which states that if the river rises 24 cubits 3½ hands at Elephantine, it will water the country satisfactorily. On this Elephantine gauge there are scales in Arabic and Greek and on the stairway Greek inscriptions, dating from the reigns of several Roman emperors, giving the year of their reigns and the high water mark of the Nile. There is evidence that the readings of the gauge during the rising river were communicated to the people below, so that notice of the coming flood was circulated. This was before the days of the telegraph. *Now* the engineers at Assuan and Assiut and Cairo receive telegraphic reports

daily from all up-river points as far as Gondokoro, 1,070 miles (1,722 km.) above Khartum, or 2,870 miles (4,621 km.) from the sea, and I saw on Mr. Watt's profile at Assuan the stage that day at all points, so that by this profile they know just what to expect as to height of the floods and the time of the same and the volume of water coming down, and can figure out mathematically and correctly how much water they must let through the sluices for irrigation.

We now come to the crux of the whole situation. Why were the four great barrages built? At Assuan, the largest and for a reservoir; Assiut, to turn some of the water coming down from this reservoir into the Bahr Yusuf (Joseph of the "coat of many colors"), now called the Ibrahimiya Canal on the west of the Nile, which is a *perennial* canal, and Esneh which supplies water to its canal in flood seasons and regulates it and this is called an "inundation" canal, since during low Nile it is dry, the bed being 6 or 7 feet (1.8 to 2.1 m.) above the level of low Nile. The summer crops of Egypt require water during the two or three months of low Nile, but nothing had ever been done until late years to supply this water, and even in modern times the delta was the only part where there was any attempt made to hold the water up and distribute it over the land. From time immemorial the Nile had been let alone to water the land of Egypt according to its own sweet will and annual caprices, and there are no records to show that there was by any ancient ruler any really serious effort to regulate the supply of water by dams and reservoirs, although there are indications that Mena, first king of the first dynasty 4,000 years before Christ built a great dyke at Memphis to control the flood and provide "basin" irrigation, which caused Egypt to flourish and become great. The deterioration of those great works led to the downfall of Egypt's greatness. Napoleon I saw the necessity to the delta of regulating the supply to the Rosetta and Damietta outlets, so as to let the whole flow first down one branch and then down the other, and in 1833 Muhammad Ali blocked the head of the Rosetta branch by a stone dam to throw the water into the Damietta branch, wherefrom, then, all the large canals of the delta took their supply. But this worked mischief to Alexandria and the eastern delta as well, and a dam was built across the head of each branch, but much bad work was done, through no fault of the engineer, Mougel Bey, who was unable to control his master, who insisted that

so many tons of concrete should be dumped every day into the foundations, whether or not the river was flowing over it. Naturally, the cement was all washed out and the bare stone of the concrete alone remained. However, the barrages were completed in 1861 on Mougel Bey's plan. These works are the famous barrage. They are 14 miles north of Cairo. The Rosetta Barrage has 61 arches or openings and two locks, and is about 1,500 feet (450 m.) long. The Damietta Barrage has also 61 arches and 2 locks and is about 1,730 feet (520 m.) long. The repairs which became necessary later led to a restoration in 1886-1891. These barrages can hold up a head of 13 feet. Poor old Mougel Bey, upon whom the Egyptian government visited the sins of his master, Muhammad Ali, was broken down with grief and almost starvation. When Sir Colin Moncrieff was planning the restoration of the barrage, the matter was brought to his notice, and he left no stone unturned until the government gave Mougel Bey an adequate pension, which placed him beyond reach of want.

This barrage is doing good work, but more water was required for the cotton crops of the delta, and the entire valley above Cairo all the way up to Assiut must have water regularly and when needed, for it is well known that some of the most important crops of Egypt can not stand a ten days' drought. The engineers studied the entire situation and all of its peculiar conditions and decided that the necessary supply of water, and at the times needed, could best and only be obtained by making a reservoir by a dam across the granite bed at Assuan. But after the plans were submitted, financial difficulties appeared by the Sudan Rebellion and the project was put aside. But there were men who saw clearly the immense increase in agricultural prosperity that an adequate reservoir of water to use at the proper time would bring about, and among these few men was Sir Ernest Cassel, who in 1898 arranged for the money to build the Assuan dam. He selected the late Sir Benjamin Baker as his consulting engineer and Sir John Aird & Co. as contractors, who undertook to complete the dam and the Assiut Barrage by December, 1903. The Egyptian Government, by the advice of Sir William Garstin, accepted the offer. The work was started in 1899 and completed in 1902, a year in advance of the time agreed upon.

The dam as originally designed would have submerged, for several months each year, the temples on the Island of Philæ, just above and within the area of the reservoir, and such was the

opposition to it by the antiquarians that the plans were modified and the height reduced. But, in 1904, it became apparent that the dam must be raised for the prosperity of Egypt, and the "heightening" was begun in the winter of 1907. The original storage capacity was 34,610,000,000 cubic feet (980,000,000 m.^{3}) of water, but the completed dam stores 88,224,000,000 cubic feet (2,500,000,000 m.^{3}) by the increased height of about 25 feet (7.6 m.). The constant supply of water, over two and a half times that of the original dam, does away with the ancient system of basin irrigation, which irrigated only a part of the year, and the cultivation of the fertile lands of Egypt can go on all the year, furnishing as it does *perennial* irrigation. When it is known that 140,000,000,000 cubic feet (3,964,000,000 m.^{3}) of water could profitably be used in bringing new and reachable areas of Egypt into cultivation, it will be seen how much yet ought to be done and must eventually be done to meet this demand of Egypt. How this increased amount is to be obtained is a problem, the solution of which can not be long postponed. No doubt, more reservoirs will be established at suitable points between Assuan and Khartum. With a sufficient amount of water at command and much of it still wasting into the sea, doing no one any good, it would seem that the brains and the money should be found. The brains are there, I am sure, for no one can examine the splendid work already accomplished by the English engineers without having entire confidence in their ability to cope with this new and possibly more difficult condition. Since writing the above, reports have reached me that another dam between Assuan and Khartum is proposed.

From the engineers I learned the following about what is called the "heightening" of the dam (and while it has, no doubt, been described in the engineering periodicals and you are therefore fully informed about the work, it seems appropriate to give a brief résumé of a rather unusual work). Sir Benjamin Baker insisted upon the thickness on the back of the dam being the same as the additional height; that is, about 5 meters (16.4 feet), also, that, as the original dam had passed through its expansion and contraction period and from green to hard masonry, it would be necessary to leave an open space of 6 inches (15 cm.) between the new and the old work until the former had accommodated itself to the effects of the large variations of temperature and had got itself into the same permanent condition as the old

masonry, and then to fill up the space solidly with Portland cement grout. This was done; and to hold the new work out from the old wall evenly and to hold it up, a plan was designed of inserting steel rods into the old masonry. These can be plainseen in Fig. 9. The addition to the dam is about equal in volume to the original dam. This dam allowed for a larger freeboard than experience had shown to be necessary, so perfect is the control of the mass and height of the water impounded, so that, by reducing this freeboard, 25 feet (7.6 m.) of additional water height was safe-

Fig. 6. Assuan Dam. Showing West Channel Work in May, 1901.

ly obtained. Another interesting feature which developed was the action of the enormous velocity and impact of the water discharged through the sluices upon the rock below the dam and even upon the sides of the sluices, which showed considerable wear. They were then lined with cast iron. A new talus below the dam 100 feet (30.5 m.) to 200 feet (61 m.) wide and of very considerable thickness in some places to make it of uniform height, was built of Portland cement concrete and granite at a cost of $1,500,000 (7,500,000 frs.). This talus is divided by cross walls perpendicular to the axis of the dam to facilitate inspection.

Raising the dam made it necessary to change the navigation locks. The height of the walls of the four original locks has been increased 14 feet (4.3 m.) to 21 feet (6.4 m.) and a fifth lock has been added at the foot of the flight. The highest gates, which are the new ones at the uppermost lock, are 84½ feet (26 m.) high, weighing about 200 tons. They are rolling gates, not rolling on sills, but on a girder that runs out with them from the recess. They are worked by hydraulic power at a pressure of about 800 pounds per square inch (56 kg. cm.). These

Fig. 7. Assuan Dam. Showing West Channel Work in June, 1901.

gates are designed to withstand a pressure of about 3,000 tons, when the reservoir is full and lock No. 1 is empty. This gate was operated for our benefit by Mr. Watt and his assistant. (These gates and machinery were furnished by Messrs. Ransomes & Rapier. The photographs, snap-shots of ours, show the mechanism fairly well. See Figs. 13 to 15.)

To summarize the changes made in the dam, which was inaugurated on December 23, 1912, by the Khedive and Lord Kitchener, who read a message of congratulation from King George; the following are the principal dimensions of the old and the raised dam:

	As built.		As raised.	
	Feet	*Meters*	*Feet*	*Meters*
Level of water above sea level	348	(106.1)	370.73	(113.0)
Pierced portion, width at top	23	(7.0)	36.1	(11.0)
Pierced portion, width of roadway	16.4	(5.0)	29.5	(9.0)
Solid portion, width at top	17.8	(5.4)	30.8	(9.4)
Solid portion, width of roadway	11.2	(3.4)	24.4	(7.4)
	Cubic feet		*Cubic feet*	
Storage capacity	34,610,000,000		88,224,000,000	

Fig. 8. Assuan Dam, showing Sluices. March, 1908.

On my way up the river certain questions to ask Mr. Watt were noted and sent on to him in advance. These he transmitted later to Mr. Lambton at the principal administration office at the Assiut Barrage and inlet to the Ibrahimiya Canal, answers to which have now been received and are given below as interesting data.

The difference in height of water surface at Assiut in low summer level and floods is 20 feet (6.1 m.). The summer level is affected by the regulating barrage in the river and in the canal nearby. Formerly it was 30.7 feet (9.4 m.). The area of the river section is 98,000 square feet (9,100 m.^{2}). The slope of

the flood surface is 1/11,800, velocity of current in flood is 4 feet per second (1.2 m.), maximum discharge is 448,000 second feet (12,687 m.^{3}). The gates are not down in high flood, the whole volume passing through.

The Ibrahimiya Canal. Width at bottom is 180 feet (54.9 m.); depth in floods is 31 feet (9.5 m.). The bed level of the canal is the same as that of the barrage. Slope of water surface in the canal is about 4.8 inches per mile (7.6 cm. km.). Length of canal to its entrance into the Fayum, 224 miles (360 km.). At this

Fig. 9. Assuan Dam. Beginning the Work of "Heightening." View taken April, 1909.

point the Gizeh Canal takes off and runs 76 miles (122 km.) farther, discharging into the Rosetta branch of the Nile some 3½ miles (5.6 km.) beyond the delta barrage above described. Maximum discharge of the canal is 25,779 second-feet (730 sec. m.^{3}). Number of acres irrigated by this perennial canal is 1,123,000 (455,000 hect.). Irrigation in the main is by gravity, but there are places where a lift is necessary. The water is raised by steam engines and pumps, by water wheels and by "Shadoofs." Drainage is by gravity, but there is also a pumping station which is used during a brief period.

The character of the crops is as follows: Cotton, sugar, wheat, barley, maize (corn), clover (two kinds), onions, beans, lentils, foungreek and lupins.

At Assiut, on passing up, a dredge was at work. The answer to my question about this was: The dredge was working for navigation purposes only. It also works when required in the Ibrahimiya Canal. The work varies, sometimes none is required in the river, but is required in the canal, and sometimes some in the river and none in the canal. The average amount in

Fig. 10. Assuan Dam. Heightening Lock No. 1. South entrance, looking East. (October, 1910.)

both places in the last five years is 55,000 cubic yards (42,000 m.^{3}).

The barrage across the Nile at Assiut for the Ibrahimiya Canal is 2,691 feet (820 m.) long, with a lock large enough for the largest tourist steamers and a swing bridge over it for the traffic across the river on the barrage, which has a wide roadway. (See Figs. 17 and 18.) There are 111 vents (openings) of 5 meters (16.4 feet) span each, arranged in 12 bays of 9 openings and 1 bay of 3 openings. Each opening has two gates, each 2.50 m. (8.2 feet) in height.

I take from "Irrigation Engineering," by Herbert M. Wilson,

M. Am. Soc. C. E., the following in regard to some features along the canal, page 122. I give the data without having the opportunity to verify them. "It irrigates about 20 large basins." "The first regulating work on the line is at Derout, where the water can be drawn from the canal by great regulating works into five branches, or passes, into the river through a large escape." "At the terminus of this canal . . . is the largest masonry escape in the world for discharging the water which comes from the canal and basins. Its maximum capacity is 80,000

Fig. 11. Assuan Dam Heightening, looking East. (May, 1911.)

second-feet (2,265 sec. m.^{3}) and it consists of 60 vents of 10 feet (3 m.) each, the maximum height on it being calculated at nearly 15 feet (4.57 m.). It is closed by great double gates operated by travelling cranes from the piers above."

Mr. Cope Whitehouse, son of Bishop Whitehouse of Illinois, had for many years a project for using the Fayum for a great regulating reservoir for lower Egypt, his idea being to restore the conditions that existed in ancient Pharaoh times, when Joseph opened the Ibrahimiya Canal (Bahr Yusuf, Joseph's Canal, for Yusuf means Joseph and Bahr means water) and let the flood waters into the Fayum, and at Low Nile out again from this

great natural reservoir into the Nile to furnish water to the delta.

The latest data I can find published gives in round figures as the population of Egypt (not including the Sudan) 10,000,000, and 10,000,000 acres (4,047,000 hect.) as the cultivable land of Egypt, provided it had water sufficient and at the right time. The cultivated land in 1910 amounted, all told, to 3,616,000 acres (1,463,400 hect.). But I am inclined to believe this an under-estimate for 1912, as the Ibrahimiya Canal irrigates now

Fig. 12. Assuan Dam Heightening. South side, looking West. (April, 1912.)

1,123,000 (455,000 hect.) and according to Mr. Macdonald there are 5,000,000 acres (say 2,000,000 hect.) in the delta, of which 2,000,000 acres (say 800,000 hect.) are now in cultivation, making in these two sections alone 3,123,000 acres (or about 1,250,000 hect.) under cultivation.

This brings me to the last problem before the engineers and the government, and that is the thorough deep drainage of the delta. Lower Egypt has now more water for irrigation and for perennial irrigation than ever before in all its history. In fact, it never had perennial irrigation until the Assuan Dam began to give it, and that can do it now, for it stores up an immense amount

of water, which is let out only when low Nile comes and during the two or three summer months. When I visited the dam on the 9th of February, 1913, there were about 850 cubic meters per second (30,000 sec. ft.) coming into the reservoir and they were letting just that much through. When the discharge falls to 500 cubic meters (17,650 sec. ft.) they begin to let the water through the sluices and draw the reservoir down to keep up the flow for

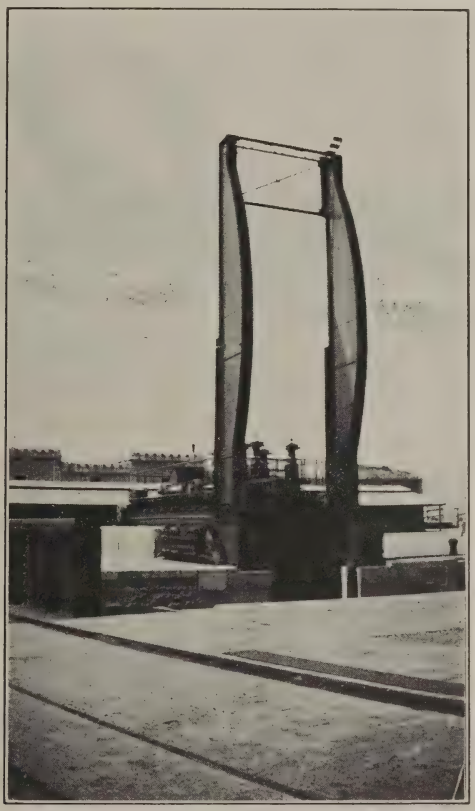


Fig. 13. Lock Mechanism of Upper Lock, Assuan Dam. View No. 1.

the Assiut Barrage to furnish the Ibrahimiya Canal with its quota and to let the remainder go on down to the Cairo Barrage. All this new water, if we can so call it "new," makes drainage imperatively necessary in the delta, especially on account of the deep roots of the cotton, which is the main and most valuable crop of the delta, there being 1,750,000 acres (700,000 hect.) in cotton of the 2,000,000 (800,000 hect.) cultivated. The roots of

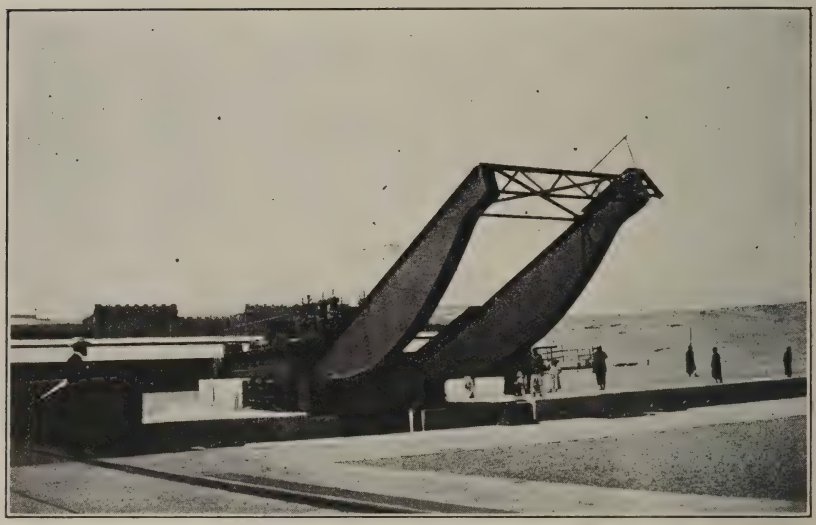


Fig. 14. Lock Mechanism of Upper Lock, Assuan Dam. View No. 2.

Fig. 15. Lock Mechanism of Upper Lock, Assuan Dam. View No. 3.

the cotton go down into the soil about 5 feet (1.5 m.) and it injures the cotton if the ends of these roots are kept too moist.*

There is now well worked up a project for draining the entire delta. Careful minute surveys have been made over the whole delta and it has been ascertained by the contour maps that the delta is divided into eight districts formed by the existing and disused branches of the Nile, for it is well known that the land is always highest along the line of outlets of sediment bearing rivers. Two of these districts are prepared for the drainage operations which are to be carried out by pumping. The first district will be the one back of Alexandria, and Lake Mariut will be entirely dried up. A large pumping plant will soon be instal-

Fig. 16. Entrance of Imbrahimiya Canal from Nile, at Assiut.

led there and will lower the level of the water and sub-soil water about 6 meters (20 feet). The second section lies about in the middle of the delta. When these have been drained, the others will be taken hold of, until the whole 5,000,000 acres (2,000,000 hect.) will be cultivable land, and thus 3,000,000 acres (1,200,000 hect.) of wonderfully fertile land will be added to the agricultural land of Lower Egypt. Accompanying the drainage of the delta there will be laid out a complete system of irrigation—main canals, secondary and ditches—all draining into the drainage system. Therefore, instead of Egypt drifting into hopeless

*In April last the Nile was reported lower than at any time in the last one hundred years, viz, 230 cubic meters per second (8,120 second-feet).

insolvency, whither it was rapidly tending a few years ago, and her people into the despair of abject poverty, Nile water, utilized and distributed with impartiality, is making, and will still further make, strong, hopeful men of her fellaheen, and is filling her exhausted treasury. The original Assuan dam has repaid its cost over and over again and the new work we have briefly described and which was put into use only a few months since, will add enormously to the productiveness and wealth of Egypt and to the happiness and general welfare of her people.

Monday, February 17, saw us off for Gibraltar, saying goodbye to our Cairo friends and the interesting and hospitable Shepheard's Hotel, and, assisted with our baggage on board the *China*

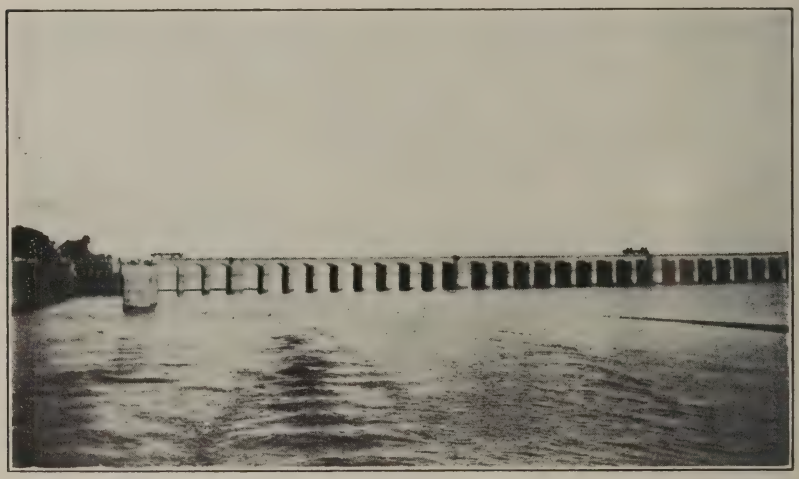


Fig. 17. Barrage across Nile at Assiut, showing Lock.

at Port Said by Mr. Koenig, the Resident Engineer of the Suez Canal, we steamed out of the harbor, taking a last look of the heroic statue of Ferdinand de Lesseps and the breakwater, and started on our six days' voyage, with one day at Marseilles, where we arrived punctually at 9 a. m. on Saturday, February 22, the weather pleasant and cool and excellent for what was before us until 5 p. m., when we were to sail for Gibraltar.

Mr. Batard, Chief Engineer of the Port, and Mr. Bourgougnon, Chief Engineer of the Marseilles-Rhône Canal (Fig. 19) now under construction, were on the dock to meet us and take us first in Mr. Batard's sea-going launch about 5 miles (8 km.) along the route of the new canal, the terminus of which is in the port of Marseilles, straight to the entrance of the tunnel now under con-

struction through the rocky ridge that separates the Bay of Marseilles from the "Etang de Bolmon," and thence along the shores of the large Etang de Berre, about 8 miles (12.8 km.) in all, and thence along the outlet of these lakes to the "Golfe de Foz," along the northern shore of that water direct to ancient Arles, following the latter part of the way the course of the Rhône. The total length, Port of Marseilles to the Rhône at Arles is somewhat less than 50 miles (80.5 km.). It is built to bring the Rhône river traffic from Lyons and above direct to Marseilles. It is being built by the French government, with some assistance from the Chamber of Commerce of Marseilles. The large Rhône barges can then avoid the dangerous route of 25 miles (40.2 km.) around

Fig. 18. Lock through Barrage at Assiut.

Cape Couronne from the mouth of the Rhône, or rather from the mouth of the St. Louis Canal to Marseilles.

Before describing the tunnel, which is the principal feature of interest, let me say that the mouth of the Rhône has an interesting history. For centuries the question of improving it for the Rhône navigation had been a matter of study, projects and efforts, until the government started to build dikes to close the smaller outlets of the delta and two jetties at the principal mouth. But two serious errors were made, the jetties were too far apart and the methods of construction were too slow. The silt and sands protruded beyond the advancing jetties, always in advance, the bar moving outwards until, weary of "chasing a rainbow," it was decided to build a lateral canal (Canal St.

Louis), about 3½ miles (5.6 km.) long to debouch into the Gulf of Foz.

The disappointment and its causes are shown, with sketches, in my "History of the Mississippi Jetties," published by John Wiley & Sons in 1880. The conditions and history were carefully examined by Captain Eads, at the time of his visit to Europe in 1874 to learn the facts about many improved river mouths, to support his arguments for the bill then before Congress for improving the mouth of the Mississippi River by means of jetties. His opponents were using the history of the mouth of the Rhône

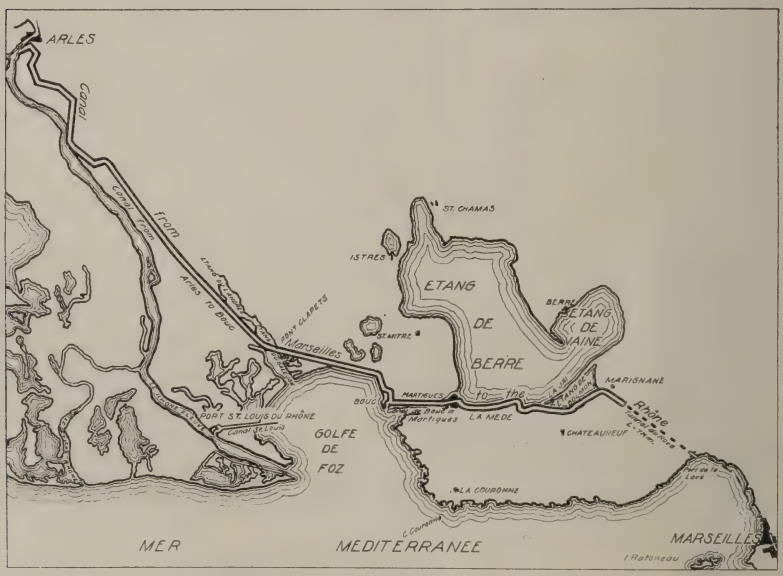


Fig. 19. Marseilles-Rhône Canal.

as an argument in favor of the Fort St. Philip Canal from the Mississippi to Lake Borgne and against the jetty proposition. Now, the Canal St. Louis is to be practically abandoned and one 50 miles (80.4 km.) long is to be built.

The tunnel is probably one of the most interesting rock tunnels ever undertaken in that it is, no doubt, the largest and one of the longest barge canal tunnels in rock. The tunnel, lined throughout its nearly 4½ miles (7.2 km.), is 22 m. (72.2 feet) wide inside the lining, and the crown of the arch (nearly semicircular) is 9.90 m. (32½ feet) high. The depth of water is to be 3 m. (10 feet). The plans illustrate the whole process of excavation, cen-

tering and lining, and are well worth the study of tunnel and canal engineers.

Briefly, as the detail plans are too extensive and numerous to be published entire, the process is as follows (see Fig. 20): A heading on the left at the base 3 m. by 3.25 m. (9.8 by 10.7 feet), then one on the right at the same level and same size, then the enlargement of the two inwards to give room for a track and

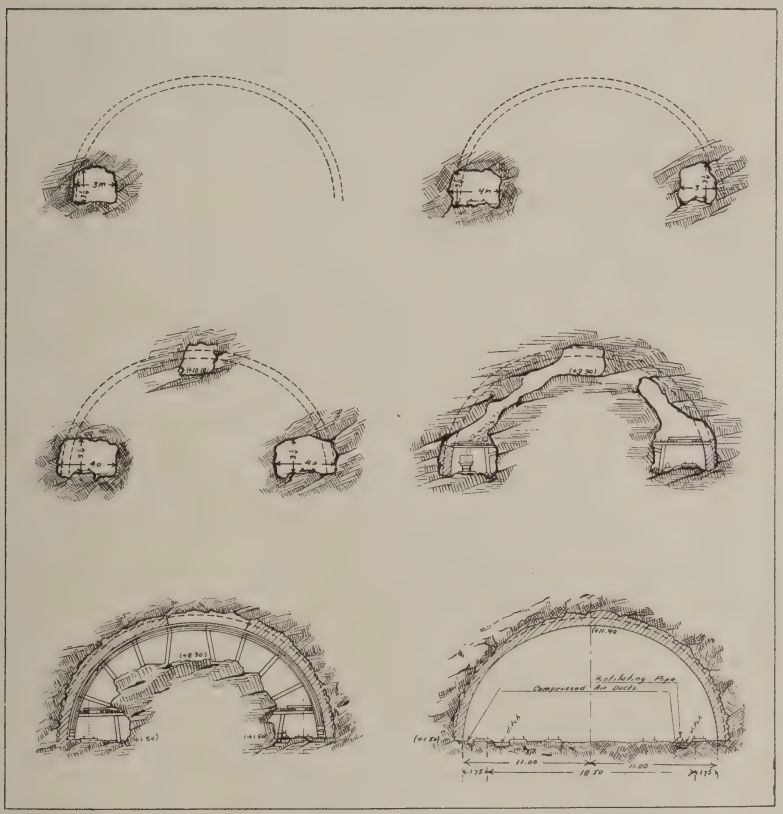


Fig. 20. Tunnel of Rhône-Marseilles Canal, showing process of excavation.

ditch for drainage, then a small heading at the summit of the arch, then the enlargement upwards of the two side headings to connect with the top heading, with a staging to receive the falling rock to be dropped into small dump cars on the track, the permanent lining at the same time started on each side, then the enlargement of the excavation under the roof on both sides, which gives a chance to put in heavy centering on short posts.

resting on the same mass core of unexcavated rock, then the masonry lining over the advancing centering completed, the centering then removed and carried forward and the bench removed. The work is done by compressed air and it has the usual ventilating machinery and pipes, compressed air pipes, drainage ditches, etc. The rock is quite varied in character. Dolomite of the Upper Jurassic, limestone, sequannian (above the Jurassic), and a mixture of the two. Suitable stone for the lining is found in the materials removed by the tunnel excavation. The "legende" in French, in much greater detail than above, is well worth study by those especially interested in such works. The contractors for the entire work are Messrs. Chagny & Co., who have built the Loetschberg tunnel and approaches between Bern and the Simplon, recently completed.

Later in the day, in company with Mr. Batard and Mr. Bouteville, we visited many parts of the port to see the operation of loading, unloading, handling and storing goods, also the enlargement of a large dry dock now under construction and a visit to the new quays in extension of the port, forming a new basin. The water is very deep and all this work is necessarily done by the use of compressed air caissons. This new basin is called "Bassin de la Madrague." In all the existing basins the quays or docks are at right angles to the line of the port. The new basin and the future basin, "Mirabeau," have piers placed diagonally to the inner quay wall of the port. The outer ends of the Madrague Basin, which is 400 m. (1,312 feet, nearly ¼ mile) long, is in 18 m. (59 feet) of water, while the outer protecting dike is in 35 m. (115 feet), one of the deepest dikes of the world. The future extension of this great dike follows the contour of about 37 m. (121½ feet) and the end of the Mirabeau dock will have to be built in 28 m. (92 feet) of water. The inner line of wall of the port now under construction is built in about 8 m. (26.2 feet) of water and extends to the mouth of the tunnel of the Marseilles-Rhône Canal, about 5 miles, with a protected opening into the sea about 2,700 m. (1.7 miles) beyond the outer pier of the Madrague Basin. The canal terminal channel in this 1.7 miles is to be about 1,000 feet (300 m.) wide and the space between it and the shore is to be filled in for commercial purposes, with streets and bridges to reach the future basins and piers of the port. The French government and the city of Marseilles are determined to hold and increase the commerce of the port, if pos-

sible, in its competition with Barcelona on the west (the port of which is being radically enlarged and improved) and Savona, Genoa, and Naples on the east. The work above outlined will necessarily cost an immense amount of money, one item alone—the tunnel with its 2½ *million cubic yards* (1,900,000 m.³) of excavation and a costly lining, is not the least item in the estimate.

Our friends delivered us to the *China* five minutes before she sailed, and on Monday afternoon, February 24, we were at the Hotel Bristol in Gibraltar. We were again among friends, for Sir H. Arthur Yorke, Royal Engineer, Engineer of the Board of Trade, London (so well known to all American railway men, as he has often been the delegate of the British government to railway congresses and is equally known to navigation men, as he is the government's member of the Permanent International Commission of the Navigation Congresses and attended, with Lady Yorke, the Philadelphia Congress, and they were with us on the excursions in the United States and Canada), knowing that we were to visit Gibraltar and that we might be debarred from seeing much that a British subject might see, wrote to Col. A. Grant, Royal Engineer in charge of all military works at Gibraltar, informing him of our expected arrival and asking his kind attention. Colonel Grant is also chairman of the Joint Sanitary Board of Military and Naval Officers, that has charge of the streets and drainage and other public improvements of the city. We also knew personally Commander Liversidge of the Royal Navy, who has charge of naval works, repairs, etc. Colonel Grant promptly detailed an engineer officer, who has charge of all works at Europa Point—barracks, houses, and batteries—to accompany us to the heights. We ascended to the height of 1,000 feet (300 m.) and would have gone to the highest point of 1,396 feet (425 m.) except for the strong wind that blew furiously that day on the heights. We did go to the end of the "Mediterranean Steps," where we could see the roof of galvanized corrugated iron which has been attached to the slopes of the mountain to catch the rainfall and conduct the water into huge tanks cut in the living rock and from thence by a tunnel under the hill to the city and the port; and yet there is a deficiency of water, which no doubt more roofing and more tanks will supply. The batteries, housing of the soldiers, the range finders and other special features I am not permitted to talk about, but it was a satisfaction to be told by Colonel Grant on the evening before

our departure that I had seen more than most people see. Anyway, the constructions, from an engineering and I may say, military, point of view were extremely interesting and the naval works as well, which are well equipped with all modern facilities —dry docks and machinery for the repair of the largest war vessels and the most delicate, like torpedo boats, submarines, etc. All this was shown us by Commander Liversidge. The extensive modern docks cost $25,000,000 (125,000,000 frs.).

After a two days' visit at Gibraltar we went over to Algeciras on our way to Southern Spain. It was a pleasant surprise to find in the Hotel Reina Christina one of the most comfortable and modern hotels in Europe and the wonder I always had had, how the members of the late Algeciras Conference could have lived in this little Spanish town, no longer existed, for the hotel outside the town, with its beautiful gardens and walks, was built for the members of this conference, although the conference room was in the municipal building in the town. The hotel Reina Christina and another good one, beautifully situated at Ronda, are owned and operated by the Bobadilla and Algeciras Railroad Co., an English company, the manager of which is Mr. Morrison, an experienced and very able railroad manager, who is also on the Board of Argentine, English and Mexican Railway Companies. I had a card to him from Col. W. Dally Jones, a retired British officer with whom I am interested in an important Swiss-Italian Railway project, and Mr. Morrison gave us a reserved compartment on the train and telegraphed his inspector at Bobadilla to attend to our transfer to the Granada train, whither we were bound.

For sixty years, since reading that beautiful classic of Washington Irving's "Tales of the Alhambra," I had longed to see it and I am glad to say there was no disappointment. The beauty and the art, the Moorish architecture and decorations are still there to delight the eye. The view of Granada below you and of the beautiful valley and plains in the distance, as far as the eye can reach from this lofty point, are most beautiful, and within sight is the point where Isabella had her quarters at Santa Fé during her siege of Granada and where Columbus came on his "recall," and almost within sight in the opposite direction, 9 miles (14.5 km.) from Granada, is the Pinos-Puente (bridge) at the foot of the Sierra, where Columbus in 1492, after having been dismissed by Isabella and had actually started to offer his pro-

ject to the king of France, was overtaken by the queen's messenger and recalled. This scene, by the way, is illustrated by a beautiful painting by the well-known American painter, Mr. Heaton, who visited the spot before executing the work. It now hangs at the head of the stairway leading to the Senate chamber at Washington, D. C.

To know that you are in the hotel, the predecessor of which and on the same spot was the home for a while of Washington Irving, and that we visit the very room near the Archives of the Alhambra given to him in which to write his "Tales," and to call up the great past and all who have lived within these walls, lends an additional interest to the beauties of buildings and country.

On March 4, we went to Seville and there we were pleasantly surprised by much that had not been anticipated and is not in the guide book. The day following our arrival we called on the United States Consul, Mr. Chas. S. Winans, who offered to arrange for a visit to the port works with Mr. Molini, "El Presidente de la Junta de Obras de la Ria del Guadalquivir y Puerto de Sevilla," to give him his position in full; but on our return to the Hotel de Madrid we found Mr. Molini's card with this note, "au nom de Mr. Brockman of Madrid," who was one of the Spanish delegates at the Philadelphia Navigation Congress and who had kindly asked Mr. Molini to call on us.

The following day in company with Mr. Winans and Mr. Molini and one of his principal assistants we made an inspection of the river in the motor launch of Mr. Molini, about 15 miles (24 km.) and then on our return a drive to see the principal work now under construction, the Alphonso XIII Canal, which is a "cut-off" that eliminates a very pronounced bend in the river just below the port.

The subject of a port at Seville and the improvements of its facilities and particularly the improvement of the river and the bar at its mouth in the sea is of sufficient importance for a complete exposition of the project, but it would lengthen too much this summary account of our travels. It is not an exaggeration to say that the commercial improvements under construction will in a few years raise Seville to an entrepôt of the first class. It is situated by the course of the river 100 km. (62 miles) from the sea. This makes it essentially a port of "penetration" into the country and for that reason one of great importance, lying, as

it does, in the midst of a rich agricultural country. Within three years vessels of 8,000 to 10,000 tons can reach Seville, and as they come from the sea will find a depth of at least 30 feet (9 m.), a channel well lighted and buoyed all the way from the sea offing to the city and, on arrival there, will find a quay of adequate length for many steamers, the full depth alongside and all of the latest facilities for cargo—cranes, sheds, and warehouses.

A brief sketch, at least, should be given of the history, physical features, the improvements and the commerce illustrated by the accompanying plans (Figs. 21 and 22). The improvements of the Guadalquivir were begun in 1794 by the straightening of one

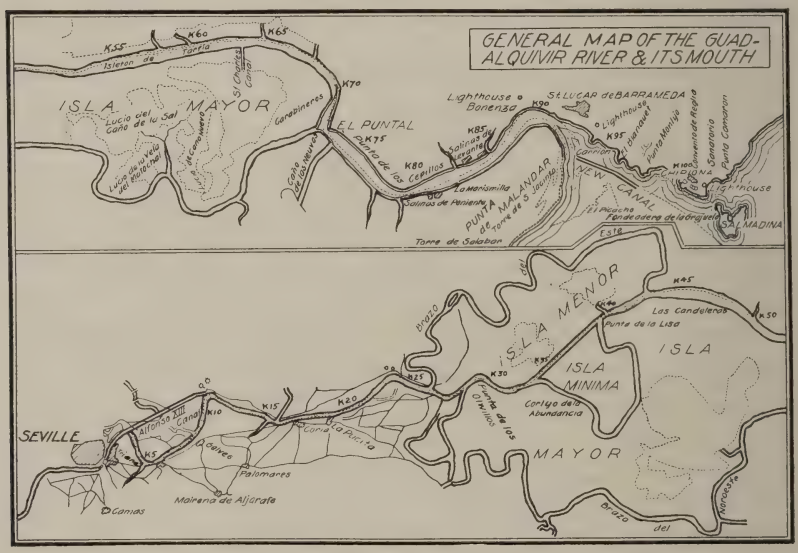


Fig. 21.

of the most pronounced bends at the Merlina cut-off. In 1816, the Ferdinanda cut-off was made. Then, up to 1863, no further work of improvement was undertaken. The draught of vessels was limited to 1.5 m. (5 feet) and the total annual traffic was 80,000 tons. Between 1863 and 1868 the state, by the insistence of the commercial interests, executed works which cost about 11,000,000 pesetas (about $2,200,000 par value), which was expended in improving the navigation of the river and building the first docks (quays). The commercial interests contributed 1,200,000 pesetas ($240,000) of the above amount. This improvement increased the navigable draught from 5 feet (1.5 m.) to 15 feet (4.5 m.) and the maritime traffic rose immediately to 108,000

tons. Between 1868 and 1871 the whole work stopped and not even were the navigation conditions maintained. At the end of 1870 there was formed the "Junta de Obras" (Board of Works), of which Mr. Molini is president and chief engineer and has been for many years. Since that date the development has been continuous up to the present time. In the year 1910 the traffic in tons of freight amounted to 1,250,152. In the maritime part of the commerce there were 1,407 vessels: 943 Spanish, 250 English, 69 Norwegian, 46 German, and 99 of various flags. There are three navigation companies domiciled at Seville, owning 43 steamers. The amount spent by the Junta, 1871-1910, is 52,710,-

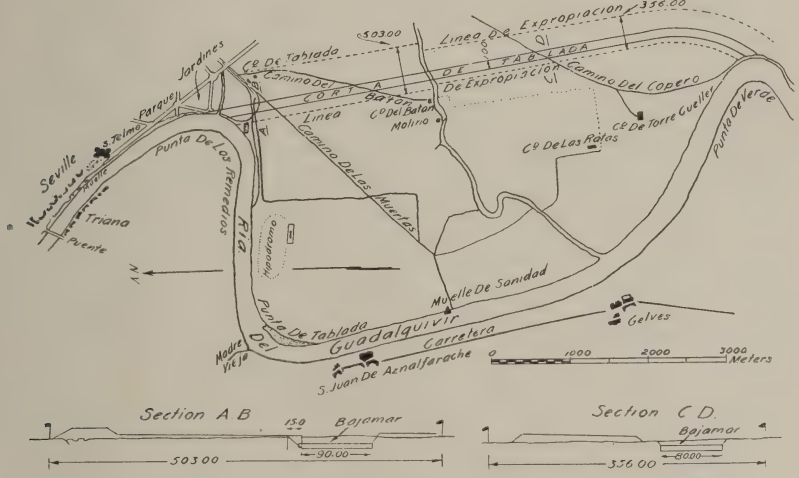


Fig. 22.

497.20 pesetas, over $10,000,000. On June 23, 1903, a general plan of improvement was approved by the government, which will be completed in the year 1916, when the larger class of vessels can reach Seville. An estimate was made by Mr. Molini of the work, which would remain to be done on December 31, 1910, on the plan above mentioned:

	Pesetas.
On improvements of the port	12,755,324.78
First section of the river	3,350,000.00
Second section of the river	350,000.00
Total	16,455,324.78
Or, about	$3,200,000.00

The "Canal de Alphonso XIII" shown on the plan (Fig. 22) and referred to before, is a separate work and will cost, all told, about 10,200,000 pesetas, say $2,000,000, and this with other rectifications of the river will materially reduce the height of the floods, which have in the past been sometimes quite disastrous by the overflow of the banks, submerging the lower part of the city. The estimate of the amount to spend annually for the present three years is as follows:

	Pesetas.
1911	1,234,000
1912	2,034,000
1913	1,604,000
Total	4,872,000
Or, about	$974,000

In doing the work, including principally now the completion of the Alphonso XIII Canal, 330 feet (100 m.) wide at the bottom, there is an excellent outfit of dredges and excavators. There are four like the one used on the Barge Canal of New York, called the *Lubecker*, which operates by an endless chain of buckets, the machine moving on rails above the water on a path of its own. Two of these can excavate to a depth of 51 feet (15.5 m.). There is a plentiful supply of locomotives and dump cars. Nearly two-thirds of the 10,000,000 cubic yards (7½ million m.^{3}) has been removed at an average cost of 9¼ cents per cubic yard (60 centimes m.^{3}). The work is being pressed day and night by the use of an electric light installation.

The entire channel from Seville to the sea will be 100 meters (330 feet) wide, and in the sea at the river mouth it is proposed to make and to buoy and light a channel 656 feet (200 m.) wide and 32 feet (9.7 m.) deep at high tide. The tide rises about 4½ feet (1.4 m.).

Every effort will be made to complete the entire work, river entrance and channel and all port facilities, before 1916, as that year has been selected for an International Exposition to be held at Seville. It will be purely an *Hispano-Americana* Exposition. It is hoped that many of the exhibits of North and South America at the San Francisco Exposition will be sent over to Seville, which can easily be done, as the exposition opens the year following that at San Francisco. It have promised Mr. Molini—and it was made at my own suggestion—that I would recommend

this to my friends in the United States. The commercial features are important to Americans as well as to Europeans.

When the navigation conditions have become what we have described, when steamers drawing from 26 to 28 feet (8.0 to 8.5 m.) and of from 8,000 to 10,000 gross tons can comfortably reach Seville, there will be an importation and exportation much beyond what has been possible hitherto. The lowering of freights to and from Seville by the increase of the tonnage of steamers, the development of coastwise traffic as well, produced by the increase in the amount of merchandise received and exported—all will tend to increase the trade of Southern Spain.

There is another important work now developing, and that is extensive irrigation between Cordova and Seville. This will result in the intensive cultivation of cereals by modern and more satisfactory methods of agriculture. During the entire year there will be sure and abundant crops, that now often fail through want of rain when it is most needed. During the summer season there is scarcely any rain, the average for the whole year is only about 23 inches (58 cm.). The work of irrigation has begun on the main canal, taking the water from the Guadalquivir. The total length, including main canal and principal branches, is over 150 miles (241 km.). The irrigated area will contain about 300 square miles (192,000 acres or 77,000 hect.) on the left bank of the river. To supply sufficient water at times of low river six reservoirs will be built, containing about 260,000,000 cubic meters of water (9,180,000,000 cubic feet). On the right bank there will be five short canals with a total length of 58 miles (93 kilom.), each with its own reservoir; the area to be irrigated being 40 square miles (25,600 acres or 10,360 hect.). The dams of the reservoirs will range from 70½ feet (21.5 m.) to 180 feet (55 m.) high. The estimated cost of the entire system is over 40,000,000 pesetas ($8,000,000). The proprietors of the lands benefited are to pay one-half and the general government one-half. A satisfactory arrangement has been made by which the property owners are to pay 10 per cent during construction and 2 per cent annually afterwards for twenty years. The government is to arrange in the annual estimates for enough funds to keep the work along. Mr. Winan's report in Daily Consular and Trade Reports No. 160, July 11, 1911, page 154, will give all necessary details. It will be seen, from the statements about

Seville, that this part of Spain is awake to the necessity of developing its commerce and agriculture.

Before leaving the subject of Seville, I wish to refer to an exceedingly interesting historical feature near by and reached by a convenient carriage ride from the city: *Italica*. It was founded by Scipio Africanus about 205 B. C. as a refuge for his veteran soldiers. In the reign of Augustus it became an independent municipality and it boasts of being the birthplace and early home of three of the Roman emperors—Trajan, Hadrian, and Theodosius. It was evidently a city of considerable importance, though now practically nothing remains of it except the ruins of the ampitheater, about 285 feet long by 200 feet wide. Considerable of it is still intact, but some terrible catastrophe or upheaval by man or nature has thrown about some enormous fragments of the stone and concrete work. One of the most interesting historical facts connected with it is that it was the home of Cornelius, the Centurion, who came to Peter after his dream—in which there was let down from heaven a sheet full of all manner of "unclean" animals, and the Lord told him to arise, slay and eat, and soon there appeared the servant of Cornelius, who, as a Centurion, was in Syria in command of 100 soldiers.

One of the interests in travelling in these old countries is unexpectedly finding facts that connect one with the past in a way to make it seem almost like the present, and this is true particularly in Egypt, until, after a while, looking upon scenes of the great and distant past that stretch back 6,000 or 8,000 years, even old Roman and Jewish times—the prophets and the birth of Christianity—seem really modern, and to an American, who calls anything old that goes back a couple of hundred years, these old civilizations make an irresistible appeal that can not be put aside.

It would, no doubt, interest you to hear the relation of many incidents of our travels elsewhere in Southern Spain, Cordova, Toledo, the Escorial and Madrid, but I will only refer to some interesting matters in Madrid, where four of our friends, who were delegates to the Philadelphia Navigation Congress, showed us many features of interest. These friends are Mr. Guillermo Brockman, Capt. Xavier de Salas, Capt. José Ma Barrera, both of the Spanish navy, and Prof. Juan Manuel de Zafra of the Royal Technical School. Through their kindness we visited the Club de Madrid, one of the finest of club buildings, and lunched

with them there and examined the building and later examined two notable structures which are being built under the charge of Mr. Antonio Palacios, one of the leading architects of Madrid. One of the buildings is a hospital for poor workmen, the funds being furnished by a wealthy lady—7,000,000 pesetas ($1,400,000). The other is a beautiful chamber of commerce, to cost

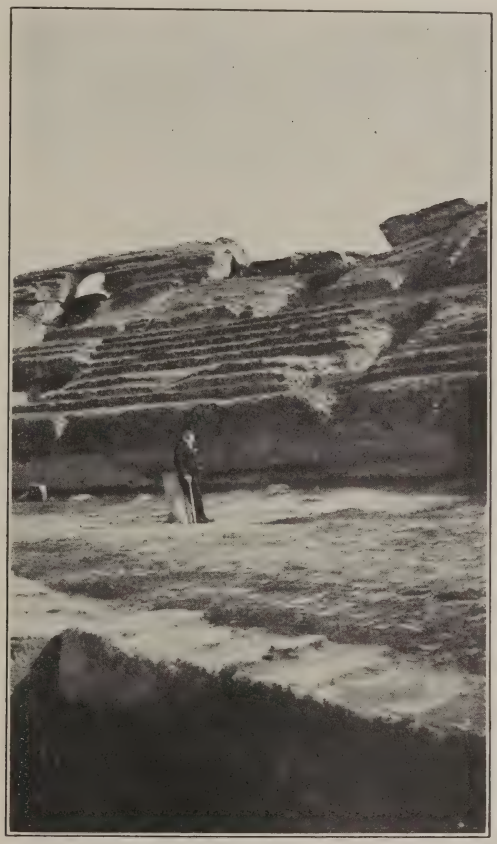


Fig. 23. Arena at Italica near Seville, Spain.

15,000,000 pesetas ($3,000,000). By the kindness of Captain de Salas we saw the game of pelota played, visited the royal palace on Palm Sunday to see the King; also the royal stables, where the array of horses, ponies, harnesses, saddles, carriages, and automobiles is very extensive and interesting; visited the house of the brother of Captain de Salas to see the crowd go by to the bull fight on Palm Sunday, a function on behalf of the widow and children of a "toreador" who had been killed in the ring two

weeks before; and, conducted by Mr. Brockman, we visited Professor de Zafra, who showed us all the features of the Royal Technical School. From a quite careful examination of it I am sure it is equal in its appointments to any technical school in the United States, although much smaller than some of ours.

Madrid is improving: better street paving, a wide boulevard being under construction to cut through the city and one built around it, very much like the new Avenida Central and the Beira Mar at Rio de Janeiro, or the Avenida de Mayo at Buenos Aires. The new Madrid Palace Hotel, where we were guests, has 800 rooms and 800 bathrooms, with telephone in each room, and in the basement is a café with 1,850 seats and a billiard room with 57 tables.

I could not close this long account of our journey without saying that on our arrival in Paris we were entertained in a most pleasant way by several of our friends of the Philadelphia Navigation Congress of 1912; after which we turned our faces toward Bern and home. It is with great pleasure that I take this opportunity to tell the story of this most interesting trip of three months. We can only wish for you, whenever you may take a similar trip, that it will prove as interesting and in every way as successful a journey as we had.

This descriptive paper may possibly convey the impression that we were looking only for engineering works of the present day; far from this. We also desired to see the interesting and instructive scenes of lands differing from our own in race, language, customs, history, architecture, art, and religion, and we found it all. The impressions are lasting. The engineering above described was incidental only to far wider and more interesting features.

Locks and Dam No. 17, Black Warrior River, Alabama.

BY

Mr. H. C. MOWER,
Assistant Engineer.

THE PROJECT.

Engineers who were interested in river improvement in 1902 will doubtless remember the very able paper on "Improvement of Black Warrior, Warrior and Tombigbee Rivers in Alabama," by Mr. R. C. McCalla, U. S. Assistant Engineer, which appeared in the April "Proceedings" of the American Society of Civil Engineers that year (Vol. XXVIII, No. 4). This treatment of lock and dam construction on canalized rivers is believed to have been one of the first exhaustive contributions to literature on the subject, and presents in detail the general conditions, methods of construction and costs of improvement on this river system up to that time.

The writer of the present paper will not presume to review the ground so thoroughly covered by Mr. McCalla, but, with a view of describing the work now under construction, the completion of which will finish the project, it might be well to glance over the work which has been accomplished on the river since the spring of 1902.

Locks 1, 2 and 3 of the Warrior River were commenced by contract in 1903 and completed in 1908 by hired labor, after which the numbering of locks was made consecutive for the entire system; Locks 1, 2 and 3 of the Tombigbee River retained their original numbers; Nos. 1 to 6, inclusive, of the Warrior River became Nos. 4 to 9, inclusive, while the locks on the Black Warrior, at and above Tuscaloosa, were designated as Nos. 10, 11, 12, 13, etc.; No. 13 being the lock of 12-foot lift which was described by Mr. McCalla as "No. 4, Black Warrior River."

Locks 14 and 15 were built in accordance with the original pro-

gram 374 and 380 miles, respectively, above Mobile, and have a lift of 14 feet each. They were started in the spring of 1908 on the same contract and were completed early in 1910. Both of these locks and dams rest on hard sandstone, the dams as well as lock walls and abutments being built of 1:3:6 cyclopean concrete with crushed stone as aggregate. The superstructures of locks and abutments are similar in design to those of Locks 4, 5 and 6, except for the increase of 4 feet in normal lift with a corresponding increase in wall sections. The gravity dams have a lift of 14 feet, are 23 feet 10 inches wide at the bases and are set into the rock about 1 foot with cut-off walls under upper and lower sides built an additional 2 feet into the rock and 4 feet wide to cut off possible percolation through surface fissures. Both dams have Ogee downstream faces; the crest curves are tangent to the sloping crest faces of 4 feet width, and the upstream faces are vertical. Operating devices, valves and irons are generally identical in design with those at Locks 4, 5 and 6 but somewhat heavier, while the same type of mitering, single skin gate is used, but designed for a 14-foot lift. In building the gates, cast iron reaction shoes were bolted to vertical channels at the quoin ends of the horizontal beams and the cast iron cylindrical posts were dispensed with. These shoes, when the gates are closed, bear on plane cast iron quoin plates set normally to the line of thrust. Oak strips bolted to the downstream face of the gate render the joint between the gate and wall water-tight. Stone-filled guard cribs were built of 10 inch by 10 inch timber on the river sides of upper approaches and similar structures, 200 feet long, were built on the bank sides of the lower approaches to serve as moorings as well as retaining walls. All cribs are provided with line hooks and snubbing posts.

The first departure from the plan outlined in Mr. McCalla's paper was made at Lock 16, 386 miles above Mobile, which was started by contract in 1909. This lock has a lift of 21 feet and was practically completed in 1911, except for six small gaps remaining in the concrete dam to serve as relief sluices. The contractor, who commenced work on Lock 17 before he had finished Lock 16, was permitted to delay closing this dam until after he had completed such portions of the walls and dam at Lock 17 as would have been submerged by backing up the pool. This effected a considerable saving in cofferdam construction. The retained percentage on the Lock 16 contract is, of course, being withheld until that work is completed and accepted.

The available length and clear width between the walls at Lock 16 are the same as those dimensions at all other locks of the system; 285 feet and 52 feet. Types of valves, special irons and operating devices are identical with those shown on drawings accompanying Mr. McCalla's article except that the gearing of maneuvering mechanisms is heavier. Gates are the same as those above described for Locks 14 and 15, except for the increase in height due to the 21 feet working head and correspondingly greater strength required in the structures.

The dam has a spillway length of 1,045 feet. It is an Ogee

Locks and Dam No. 17, Black Warrior River, Alabama. Dam Foundation showing Coal and Fire-Clay Seam. June, 1913.

type gravity structure built of 1:3:6 cyclopean concrete, about 31 per cent of the volume being derrick stone. The upstream face is vertical, the crest width is 4 feet 10 inches and the crest and roll curves are tangent to the downstream slope of 1 on 1. The dam is built about 3 feet into hard sandstone. The guard of the river wall is 12 feet, while that of the abutment is 15 feet.

Four concrete guard walls on the river side of the upper approach and a concrete retaining wall on the bank side of the lower approach take the places of cluster piles and timber cribs which guard the approaches of all other locks on the river. The

lock walls, except for their greater height and corresponding increase in width, are similar to those at locks in the system of lesser lift, and all concrete structures rest on sandstone.

Fishways or ladders have been built at Locks 1, 2, 3, 4, 5, 6, 14, 15 and 16. They are timber structures resting on sloping steps built of concrete on the river sides of the river walls, and lead on a 5 on 1 slope from 1.5 feet below the dam crests to 1.5 feet below the lower pool levels. Fishways vary in section with the lift of dam; that at Lock 16 being 5 feet 8 inches wide and 4 feet 5 inches deep. The flume is divided into 8-foot pools by 2-inch transverse timber bulkheads, through the bottoms of which square openings are cut alternating on right and left sides of successive bulkheads. From 12 inches square in the bottom bulkhead, the openings increase 6 square inches in area with each succeeding bulkhead or baffle face approaching the head. This arrangement of bulkheads results in a flow of water over each weir in addition to the baffled flow through the staggered apertures. Money is now available for the construction of fishways at all locks of the system where they have not already been built.

LOCKS AND DAM NO. 17.

Locks and Dam No. 17 are located on Squaw Shoals about one and a quarter miles above Lock 16, where the river, about 1,100 feet wide, flows between steep banks, as shown by topography on Fig. 1.

The contract for the construction of a 21-foot lift lock and dam, to be built at this site, was let to B. H. Hardaway, of Columbus, Ga., on September 20, 1910. The plans for this contract were exact duplicates of those for Lock 16 and the construction of a lock and dam of 42-foot lift 20 miles above this site, to be known as Lock 18, had been recommended by the district officer to complete the project.

In August, 1911, Congress, by special act, authorized the Secretary of War to enter into a supplemental agreement with the contractor to build a dam of 63-foot lift to be mounted by two locks in flight, at Lock 17 site. This change facilitates the development of water power by combining the lifts of the two locks at a somewhat increased initial cost, but reduces the cost of operation.

At that time the contractor had completed his supply track 15 miles long from Searles, Ala., to the reservation and had built

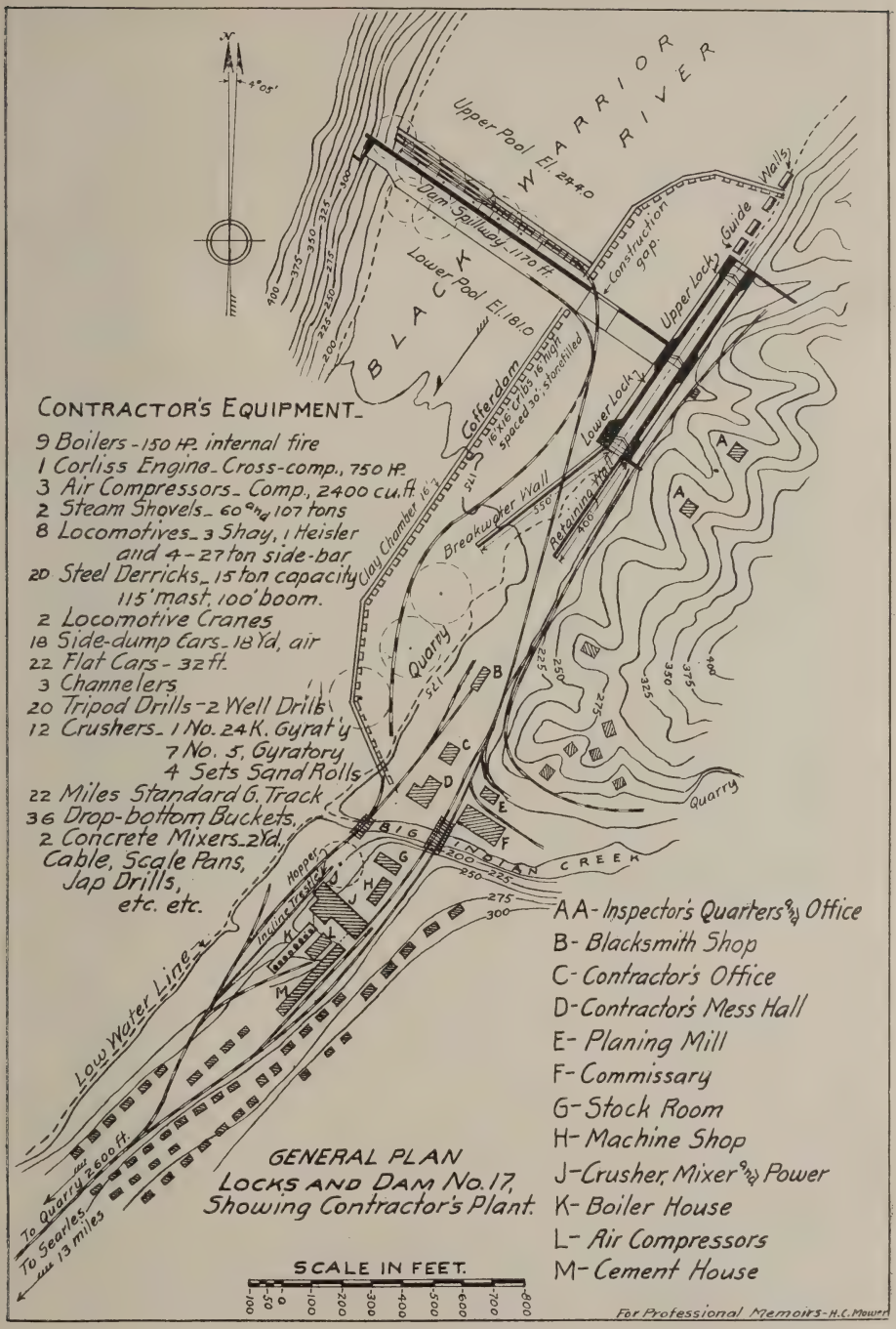

WARRIOR RIVER
Upper Pool El. 249.0
Dam Spillway - 1170 ft.
Lower Pool El. 181.0
BLACK
Construction gap
Upper Lock
Guide Walls
Lower Lock
Retaining Wall
Breakwater Wall
Clay Chamber
Quarry
Quarry
BIG INDIAN CREEK
Hopper
Incline Tramway
Low Water Line
To Quarry 2600 ft.
To Searles 13 miles
CONTRACTOR'S EQUIPMENT
9 Boilers - 150 HP. internal fire
1 Corliss Engine - Cross-comp., 750 HP.
3 Air Compressors - Comp., 2400 cu. ft.
2 Steam Shovels - 60 3/4 yd 107 tons
8 Locomotives - 3 Shay, 1 Heisler and 4 - 27 ton side-bar
20 Steel Derricks - 15 ton capacity 115' mast, 100' boom.
2 Locomotive Cranes
18 Side-dump Cars - 18 Yd. air
22 Flat Cars - 32 ft.
3 Channelers
20 Tripod Drills - 2 Well Drills
12 Crushers - 1 No. 24 K. Gyrat'y
7 No. 5, Gyratory
4 Sets Sand Rolls
22 Miles Standard G. Track
36 Drop-bottom Buckets.
2 Concrete Mixers - 2 Yd.
Cable, Scale Pans, Jap Drills, etc. etc.
AA - Inspector's Quarters & Office
B - Blacksmith Shop
C - Contractor's Office
D - Contractor's Mess Hall
E - Planing Mill
F - Commissary
G - Stock Room
H - Machine Shop
J - Crusher, Mixer & Power
K - Boiler House
L - Air Compressors
M - Cement House
GENERAL PLAN
LOCKS AND DAM NO. 17,
Showing Contractor's Plant.
SCALE IN FEET.
For Professional Memoirs - H.C. Mowry

Fig. 1.

about 17 per cent of his cofferdams; the Government had purchased valves, operating gears, special irons and the steel for gates. About half of the steel and castings are being utilized in the revised design and the balance is being held as stock.

The quantities and prices of the original contract and of the supplemental agreement follow:

Original Contract—Estimate.

7	acres	Grubbing and clearing	at $100.00	$700.00
60,000	cubic yards	Common excavation	at .50	30,000.00
12,500	cubic yards	Rock excavation	at 2.50	31,250.00
500	cubic yards	Stone filling	at 3.00	1,500.00
47,000	cubic yards	Concrete	at 8.25	387,750.00
700	cubic yards	Riprap, hand placed	at 4.00	2,800.00
4,000	feet B. M.	Miter sills	at 200.00	800.00
11,000	feet B. M.	Frames, timber, heart	at 80.00	880.00
112,000	pounds	Placing valves and spec. irons	at .08	8,960.00
120	squares	Channeling	at 25.00	3,000.00
		Cofferdams, complete		25,000.00
		Locktender's house, complete		4,000.00
		Total		$496,640.00

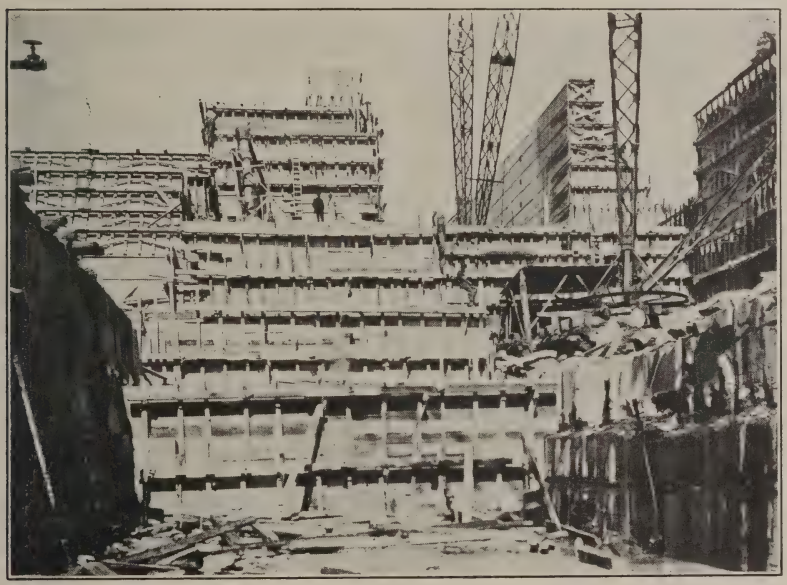
Supplemental Agreement—Estimate.

15	acres	Grubbing and clearing	at $100.00	$1,500.00
148,600	cubic yards	Common excavation	at .50	74,300.00
154,000	cubic yards	Rock excavation	at 2.00	308,000.00
500	cubic yards	Stone filling	at 3.00	1,500.00
315,000	cubic yards	Concrete	at 7.25	2,283,750.00
1,500	cubic yards	Riprap, hand placed	at 4.00	6,000.00
3,800	feet B. M.	Miter sills	at 200.00	760.00
38,816	feet B. M.	Framed timber, heart	at 70.00	2,717.12
362,553	pounds	Placing valves and spec. irons	at .04	14,502.12
		17 per cent cofferdams, complete original	at 25,000.00	4,250.00
		Cofferdams, complete supplemental		30,000.00
300	squares	Channeling	at 25.00	7,500.00
12,925	pounds	Twisted steel rods, set	at .08	1,034.00
310	linear feet	Placing tile gauges	at 1.25	387.50
		Locktender's house, complete		4,000.00
		Total		$2,740,200.74

The quantities and prices shown above under the supplemental estimate are those on which the latest agreement was based, and now obtain. This later agreement was entered into to conform to certain changes in foundations which were found to be necessary and which will be explained later.

In addition to the above contract, the completed locks and dam will cost the United States about $350,000.00, which will have been expended in purchase of site, purchase of submerged land above the dam, engineering and superintendence, telephone line, building valves and special irons, constructing and erecting gates, purchase of operating machinery and the purchase and installation of a temporary power plant.

The location of the locks and dam was established after core borings had been taken in the river bed and hill sides under three tentative locations, and estimates, based on each location, had been



Locks and Dam No. 17, Black River, Alabama. Upper Walls and Dam from Lower River Wall Foundation. September, 1913.

developed. The borings show a hard sandstone surface across the river bed, rising to a berm about 20 feet above river bottom on each side where it is covered by steep banks of slightly disintegrated rock. A seam of coal varying from 0.6 foot to 1.4 feet thick is found from 8 to 12 feet below the river bed, and a thin stratum of fire clay and slate underlies the coal.

On the surface seam of sandstone have been built all concrete structures which will not be subjected to a heavy head of water and whose functions do not necessitate disturbing the 8 or 10 feet

of surface rock. Subsequent developments in the problem of foundations are described later.

The concrete construction, in general, consists of two lock chambers in flight, each capable of lifting 31.5 feet, with wing walls at upper and lower ends which run into the rock slopes of the hillside, four guard walls on the river side of the upper approach, the dam, the abutment, the breakwater and retaining walls on the river and bank sides, respectively, of the lower approach.

The lock chambers have an available length of 285 feet, a clear width between walls of 52 feet and a minimum depth of 6.5 feet of water over the middle and lower miter sills. There will be 13 and 14 feet of water, respectively, over the breast and upper miter walls at normal stage. This depth is provided so that the service of the locks will not be interrupted should the upper pool, for any reason, be drawn off not exceeding 6.5 feet below the dam crest.

Should such a condition arise, the upper chamber will not contain sufficient water to fill the lower chamber to normal level, and the depth over the middle miter wall will be 6.5 feet minus half the draw-off, when the middle gates are opened. Water will have to be admitted through the upper valves when this occurs, to raise the level in both chambers before the interlockage can be made; but since this contingency is only a possibility incidental to the future development of water power at this site, it was not thought best to lower the elevation of the middle miter sill 3.25 feet and increase the height of the middle gates.

The lower ends of the upper lock walls were built from footing elevation 158 ± to coping elevation 258, making the monoliths 100 feet high.

The dam is placed opposite the middle gates, thereby loading the outside of the upper river wall and raising the overturning fulcrum to the elevation of the upper floor, 26 feet above the foundation: also, the dam, on this location, carries the thrust from the middle mitered gates, which is transmitted through the river wall.

The coping of the upper lock walls is 14 feet above the elevation of the dam crest. The present spillway, 1,170 feet long, in discharging the estimated extreme flow of 140,000 feet per second, will require a guard of 12 feet, the intention being to have at least 2 feet of walls and gates above water at all times. The additional 2-foot guard was added to the walls in view of the possible future construction of a power

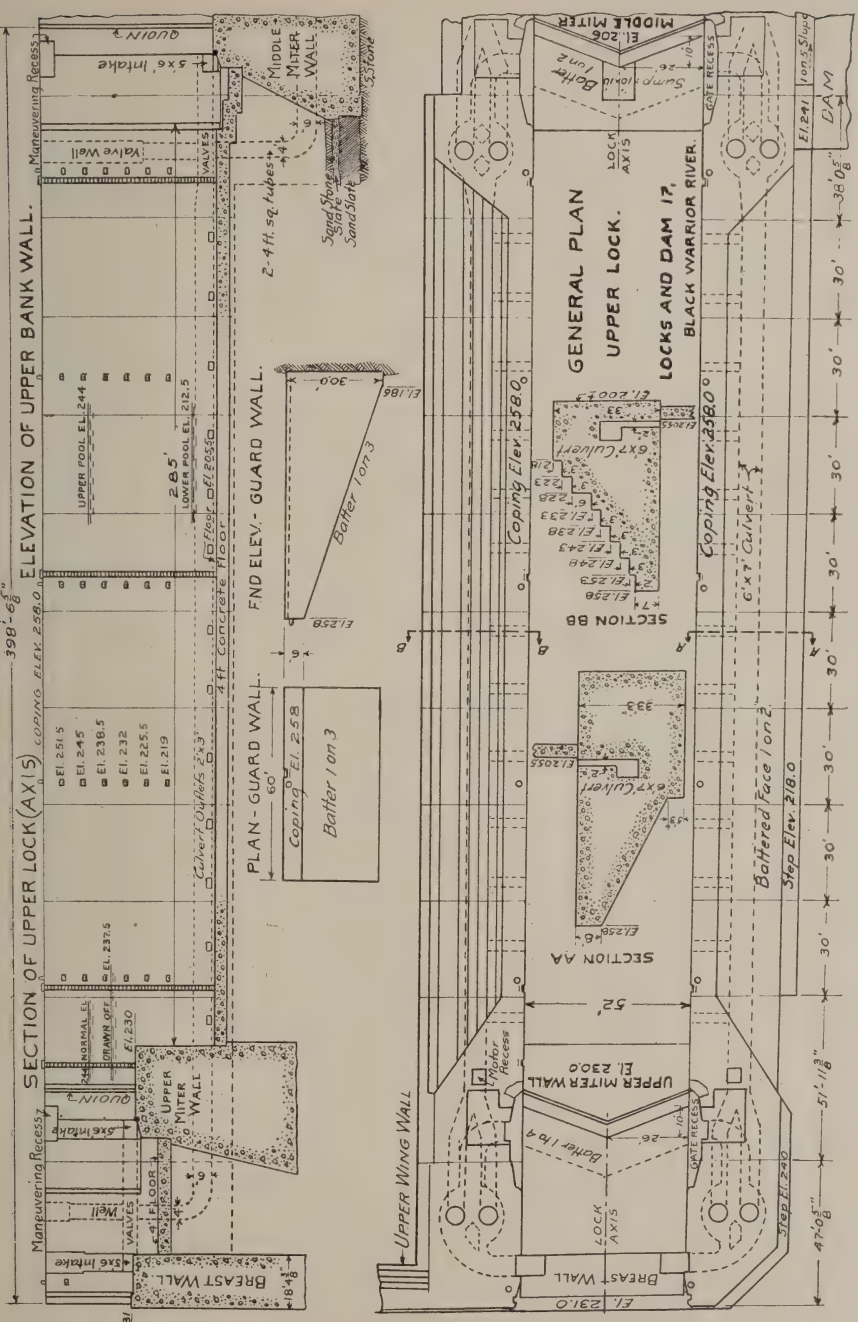

SECTION OF UPPER LOCK (AXIS)
ELEVATION OF UPPER BANK WALL.
PLAN - GUARD WALL.
END ELEV. - GUARD WALL.
GENERAL PLAN
UPPER LOCK.
LOCKS AND DAM 17,
BLACK WARRIOR RIVER.
SECTION AA
SECTION BB
DAM
UPPER WING WALL
UPPER MITER WALL
BREAST WALL
MIDDLE MITER
Batter 1 on 3
Batter 1 on 4
Batter Face 1 on 2
Coping Elev. 258.0
Coping Elev. 238.0
Step Elev. 218.0
Step El. 240
6'x7' Culvert
6'x7' Culvert
4'x6' Intake
Valve Well
Maneuvering Recess
Gate Recess
Motor Recess
Lock Axis
Sump
Drain
Sand Stone
Stone
Concrete
Rock
Earth
398'-6 7/8"
285'
47'-0 5/8"
51'-11 3/8"
38'-0 5/8"
52'
60'
30'
26'
33'

Fig. 2.

house on the dam site, which will reduce the length of spillway to approximately 1,000 feet and increase the crest head to about 12 feet.

The coping of the lower lock is 43.5 feet above normal lower pool and 12 feet above the half-lift stage. In order to prevent water from completely filling the lower chamber and spilling over the lower gates, in the case of a fouled middle valve, leak in the middle gates or careless manipulation of flow, six 3 by 4 foot openings through the river wall 4 feet below the coping are provided as relief spillways.

A system of 24-inch pipe with valves connecting upper pool, upper chamber, lower chamber and lower pool is installed in the river wall as a means of compensating for leaks in gates and valves. The four 24-inch disc valves are placed in a well which is accessible from the dam passageway, and are operated by hand power hoists on the coping. By means of this system water from the upper pool may be discharged directly into upper or lower chamber or into the lower pool; water may be drained from upper chamber into lower chamber or pool and the lower chamber can be drained to lower pool level independently of operating valves and culverts.

Culverts, with floors 6 inches above the elevations of chamber floors, are continuous through the walls from the screened intakes above the upper gates to the discharge outlets below the lower gates. The section of culverts is 6 by 7 feet except in valve approaches and draft tubes, as shown on drawings (see Figs. 2, 3, 4, and 5), and the chambers are filled and emptied by the same culverts through lateral openings of 6 square feet sectional area; thirteen laterals from each culvert opening into each chamber 6 inches above floor level. The discharge through these culverts is controlled by twelve cylindrical valves which will be described later. The valves are installed in pairs; two in each wall above the upper gates control the flow from the upper pool into the upper chamber, two in each wall above the middle gates drain the upper chamber into the lower, and the lower chamber is drained by two valves in each wall into the lower pool.

Concrete retaining walls normal to the lock axis, connect the upper and lower ends of the bank wall with rock in the hillside, and the area enclosed within the bank and these wing walls will be backfilled flush with the copings and sodded. Concrete steps connecting the upper and lower lock copings, and all edges of wall copings

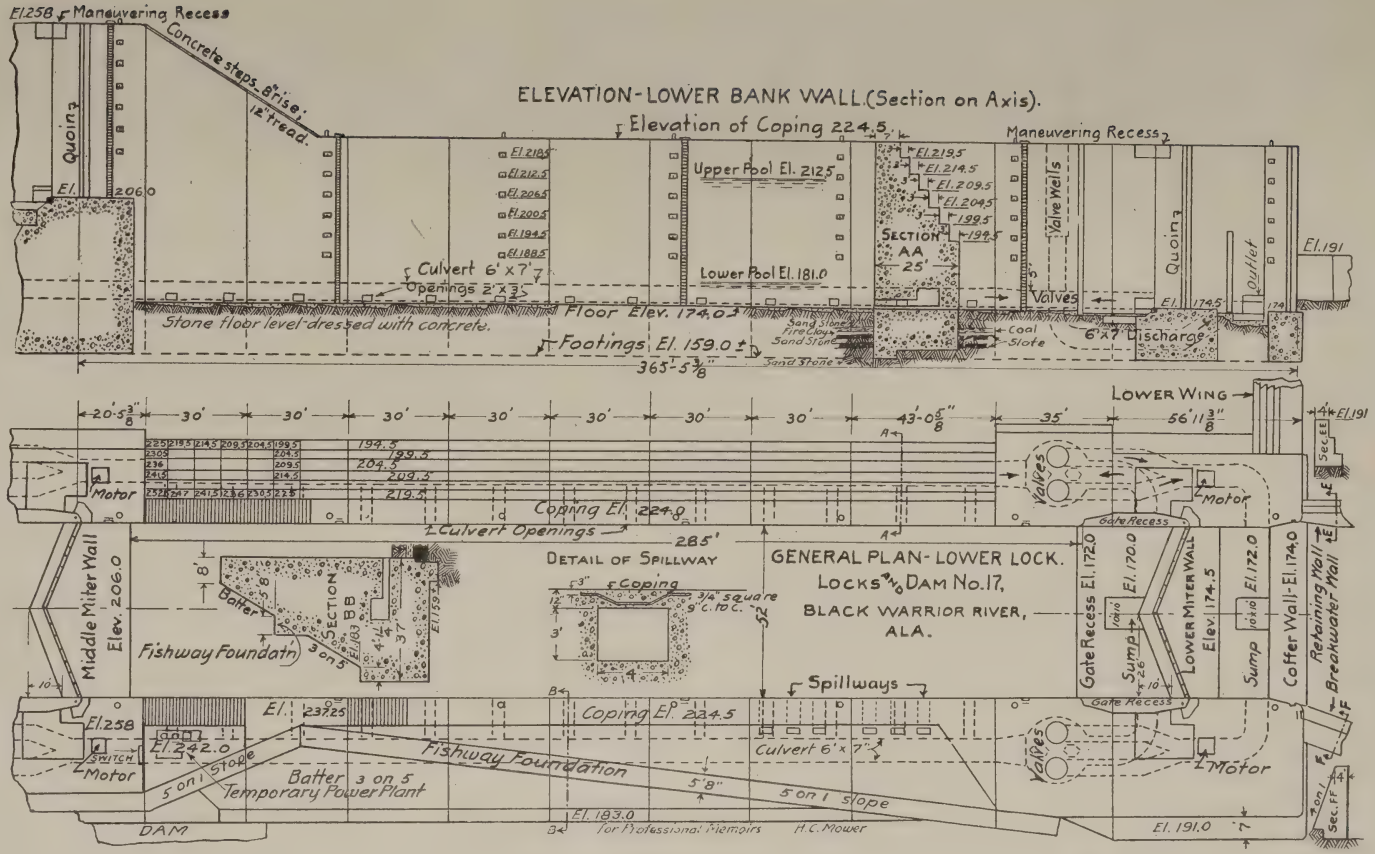

ELEVATION - LOWER BANK WALL (Section on Axis).
El. 258
Maneuvering Recess
Concrete steps, 8" rise, 12" tread.
Quoin
El. 206.0
Elevation of Coping 224.5
Upper Pool El. 212.5
Lower Pool El. 181.0
Culvert 6' x 7'
Openings 2 x 3
Stone floor level-dressed with concrete.
Floor Elev. 174.0
Footings El. 159.0 ±
365'-5 3/8"
Maneuvering Recess
Valve Wells
Quoin
Outlet
El. 191
Valves
6' x 7' Discharge
Coal Slate
Sand Stone
Fire Clay
Sand Stone
Sand Stone
SECTION AA
25'
El. 219.5
El. 214.5
El. 209.5
El. 204.5
El. 199.5
El. 194.5
El. 188.5
LOWER WING
56'-11 3/8"
43'-0 5/8"
35'
20'-5 3/8"
30'
Coping El. 224.0
Culvert Openings
285'
DETAIL OF SPILLWAY
Coping
3/4" square 9" c. to c.
GENERAL PLAN - LOWER LOCK.
LOCKS & DAM No. 17,
BLACK WARRIOR RIVER,
ALA.
Middle Miter Wall
Elev. 206.0
Motor
Batter
Fishway Foundation
3 on 5
SECTION BB
Spillways
Gate Recess El. 172.0
Sump
Lower Miter Wall
Elev. 174.5
Coffer Wall - El. 174.0
Retaining Wall
Breakwater Wall
El. 258
El. 242.0
Switch
Motor
Batter 3 on 5
Temporary Power Plant
5 on 1 slope
Fishway Foundation
Culvert 6' x 7'
5'-8"
El. 183.0
DAM
El. 191.0
Sec. E
Sec. F
For Professional Memoirs H. C. Mower

Fig. 3.

and machinery recesses are guarded by pipe handrails except on the chamber faces where rail would interfere with the handling of lines.

Tile gauges, reading to tenths, are set into the river walls above and below each of the three gate recesses. They are formed of ¾ inch square blue and white tile, blue figures on white ground, and refer to the lower miter sill elevation as zero.

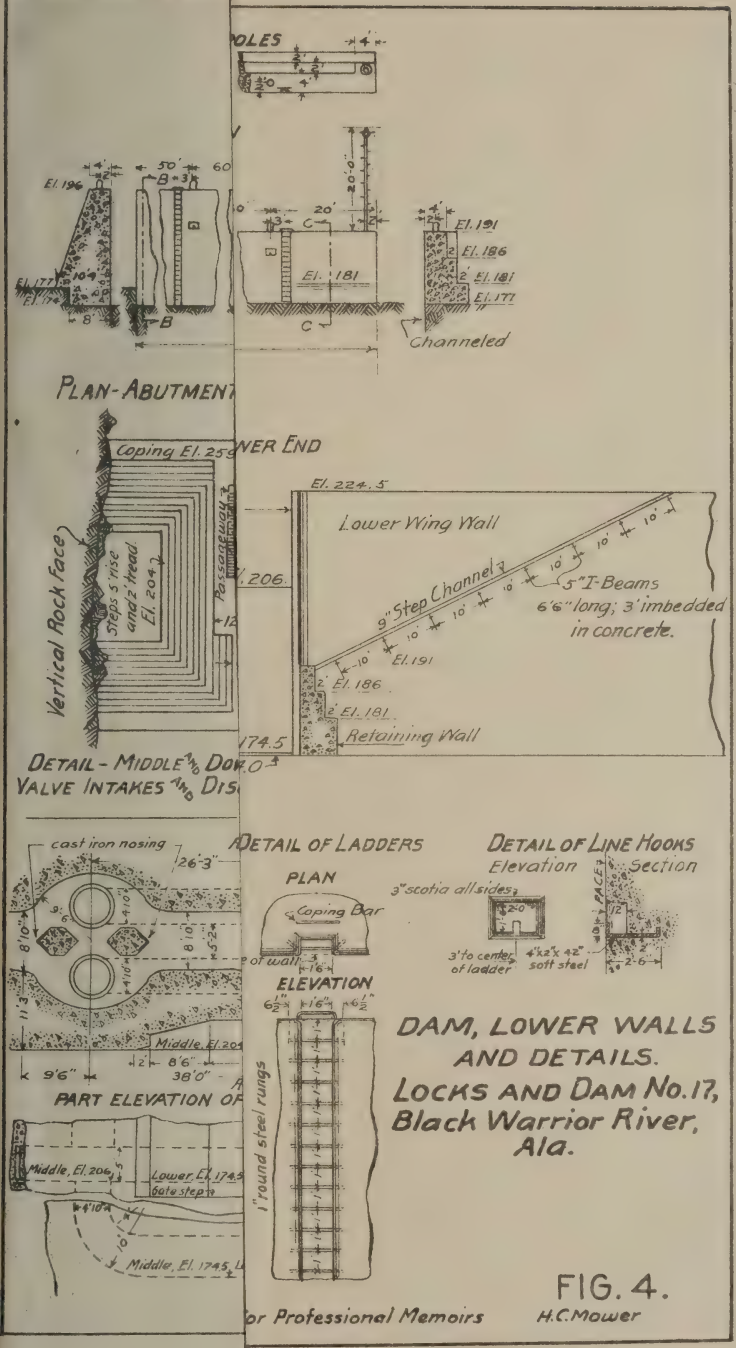
The distribution and details of ladders, line hooks and snubbing posts are shown in the figures.

The four *guard walls* are built to the elevation of the upper river wall and their vertical channel faces are aligned therewith. They are 60 feet long and spaced 10, 90, 180 and 280 feet, respectively, upstream from the end of the river wall. The coping is 6 feet wide and the back or river side faces are battered 1 on 3. A snubbing post, set of line hooks and ladder are built into each guard wall.

The dam, as originally designed, was to have rested on stripped surface rock with a cut-off wall under the upstream side sunk to elevation 163 ± to cut off possible seepage through the coal seam at about elevation 168.0. With this footing, which is indicated by dotted line on sketch of dam section (see Fig. 4), the structure, under extreme head, had the following factors of safety: 6.1 against overturning, disregarding upthrust; 2.5 against overturning, considering the full head acting over two-thirds of the base; 8.2 against shearing off the cut-off wall and sliding on the base, one-third of the horizontal section of concrete being considered as effective in resisting shearing at rock elevation. The crest curve of 17 feet radius will break into the nappe of a 12-foot head river discharge so the water will cling to the concrete face, and the roll curve of 26 feet radius will, it is thought, deliver the discharge into the lower pool without excessive turbulence and vibration.

A passageway 2 feet 6 inches wide by 7 feet high leads through the dam and connects at either end with chambers in the lock and abutment walls and thence to the copings by flights of steps. This passageway, which is drained to the downstream slope by 2-inch weepers spaced 4 feet apart, will serve as a convenient means of crossing between abutment and lock sides of the river, as a transmission gallery for electric conduits, should power ever be developed on the abutment side, and in the lock end of the passageway are located the hand wheels which operate three 4-foot square gate valves which control the waste sluices through the dam 44 feet below the crest level.

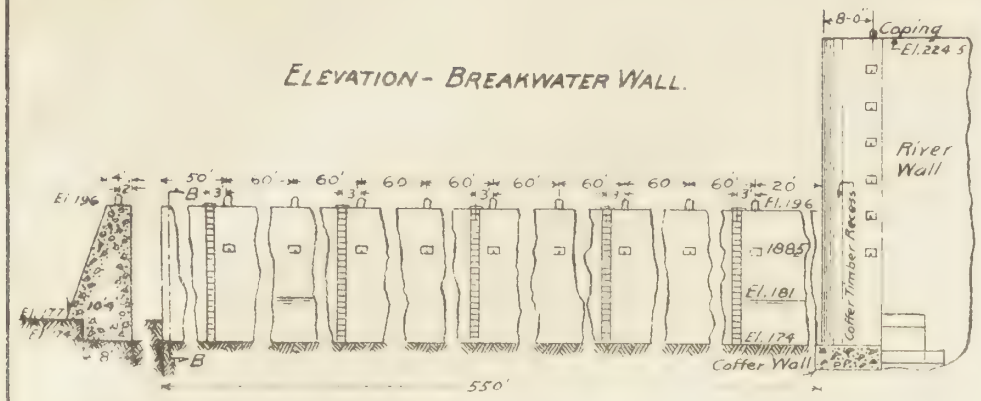
Seventy feet of the dam, adjacent to the abutment, is built on a

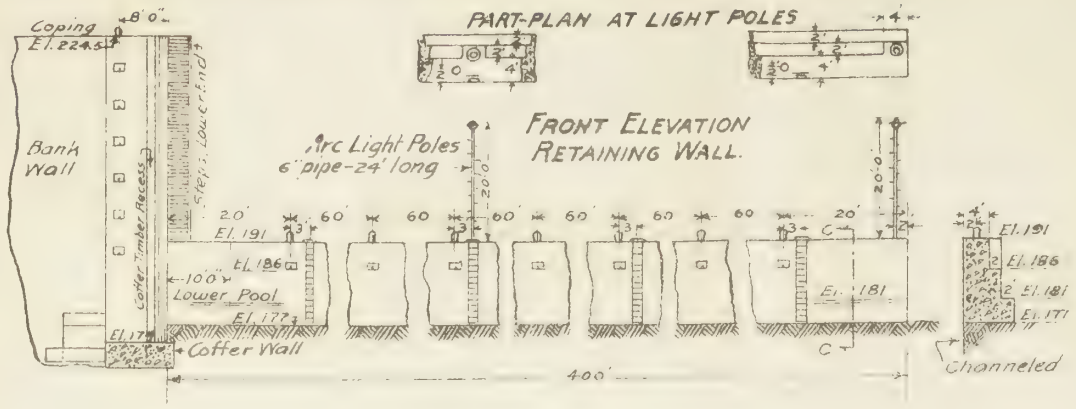


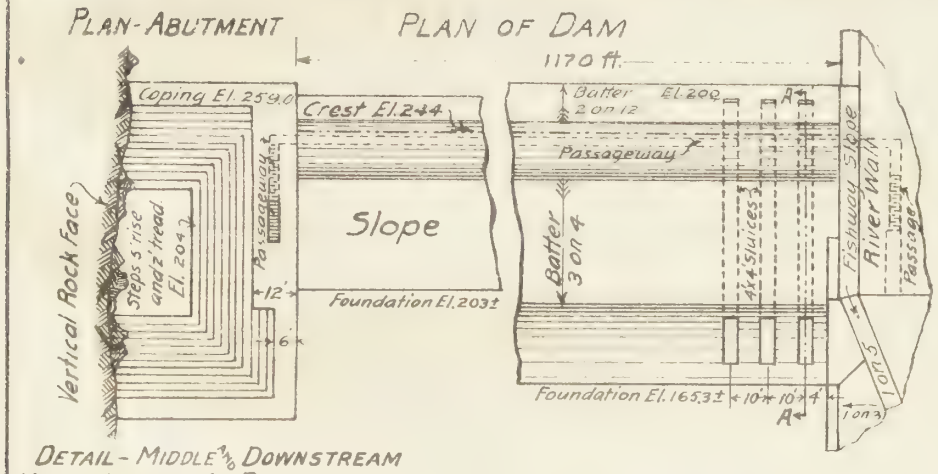
DAM, LOWER WALLS AND DETAILS.
LOCKS AND DAM No. 17, Black Warrior River, Ala.

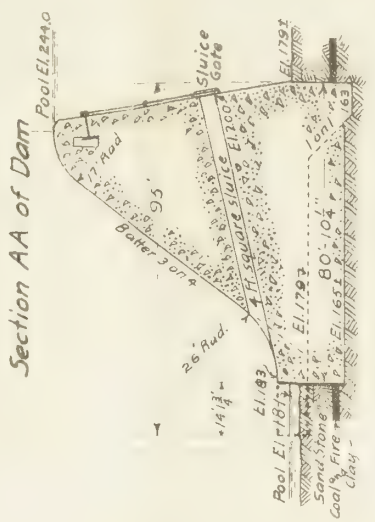
FIG. 4.

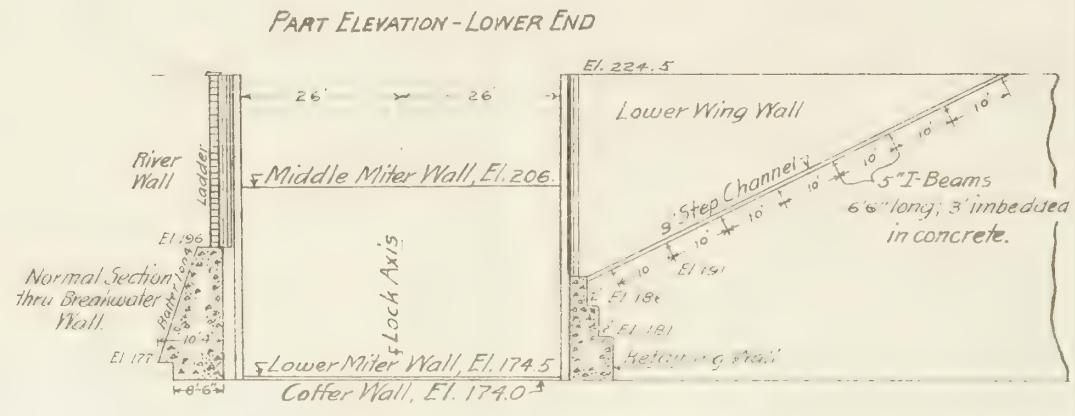
or Professional Memoirs H.C. Mower

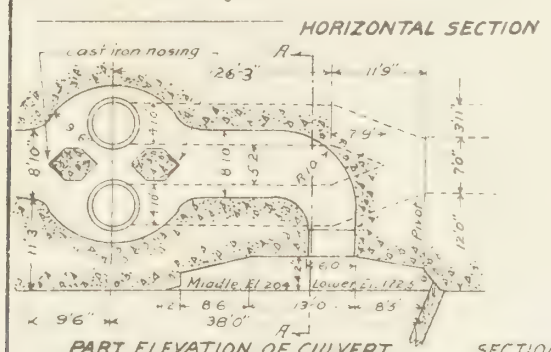


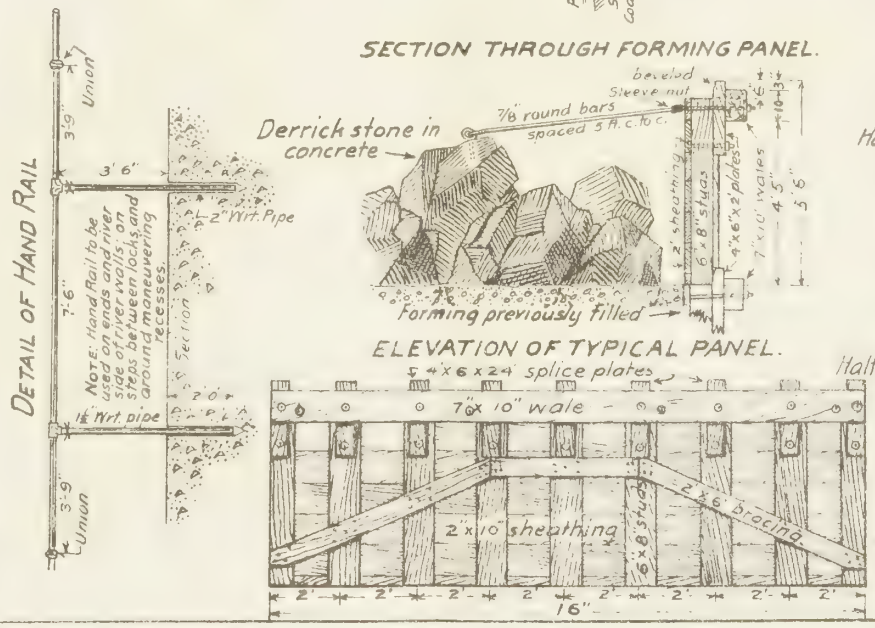


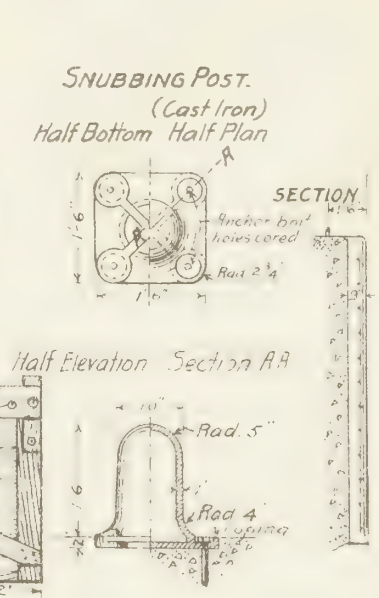


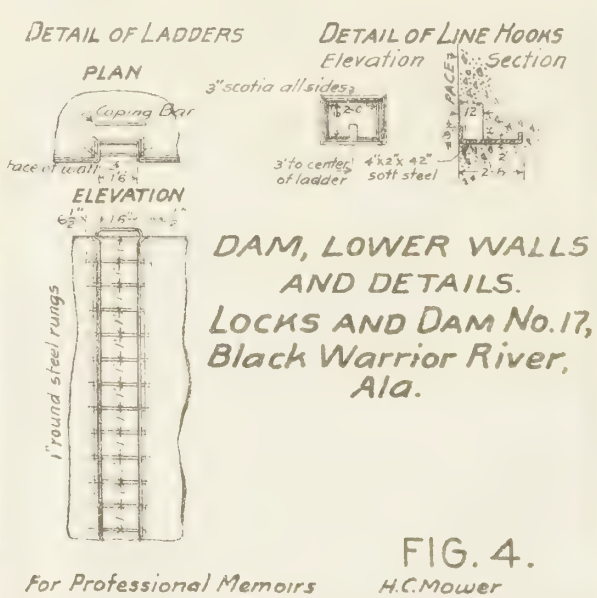







DETAIL OF HAND RAIL
Union
1¼" Wrt. Pipe
Note: Hand Rail to be used on ends and river side of river walls, on steps between locks, and around maneuvering recesses.
SECTION THROUGH FORMING PANEL.
Derrick stone in concrete
7/8" round bars spaced 5 A. C. to c.
beveled Sleeve nut
2" sheathing
6"x8" studs
4"x6"x2 plates
7"x10" wales
Forming previously filled
ELEVATION OF TYPICAL PANEL.
5 4"x6"x24" splice plates
7"x 10" wale
2"x10" sheathing
6"x8" studs
2"x6" bracing
16"





DAM, LOWER WALLS AND DETAILS. LOCKS AND DAM No. 17, Black Warrior River, Ala.

FIG. 4.

For Professional Memoirs H.C. Mower

solid ledge of rock 23 feet above river bottom, no change in the dam section being made above that elevation.

After the curtain wall trench had been excavated for part of the dam and the coal and fire clay was exposed in the channeled face, it was found that voids existed below the fire clay in places which were not shown by the borings, and that the quality of coal was not suitable foundation material under a structure of this kind. The coal was fractured on cubical cleavage planes and disintegrated readily upon exposure, while the fire clay was chalky and suggested the possibility of blowing or cutting out under a head of over 60 feet.

These developments were reported and an officer from the office of the Chief of Engineers, with the Division Engineer and the District Officer, inspected the work in the field, considered several footing sections with their respective estimates and decided to carry the dam down through the coal and fire clay to the rock below. The apron was omitted below the dam and the above decision as to footings was also applied to the lower river and middle miter walls. This change increased the rock excavation and concrete well beyond the allowable percentages and a new agreement was made with the contractor to cover this increase. The quantities and prices of this agreement are those shown before.

It was directed that after the excavation to the new footing elevation had been completed, new borings be taken in the dam foundation and tests made for by-passes before concrete should be placed. Accordingly thirty holes were drilled, filled with water, and air pressure was applied to one hole at a time. Although two holes discharged sulphur water, only one by-pass was found between one pair of holes and this was stopped by lowering the foundation of that monolith 2 feet.

In altering the foundation section of the dam, no change was made in the superstructure, and the 70-foot length of the structure on the abutment end was not changed as it was not deemed advisable to remove 30 feet, more or less, of solid rock and replace this volume with concrete in order to cut off possible percolation through the questionable seam above described.

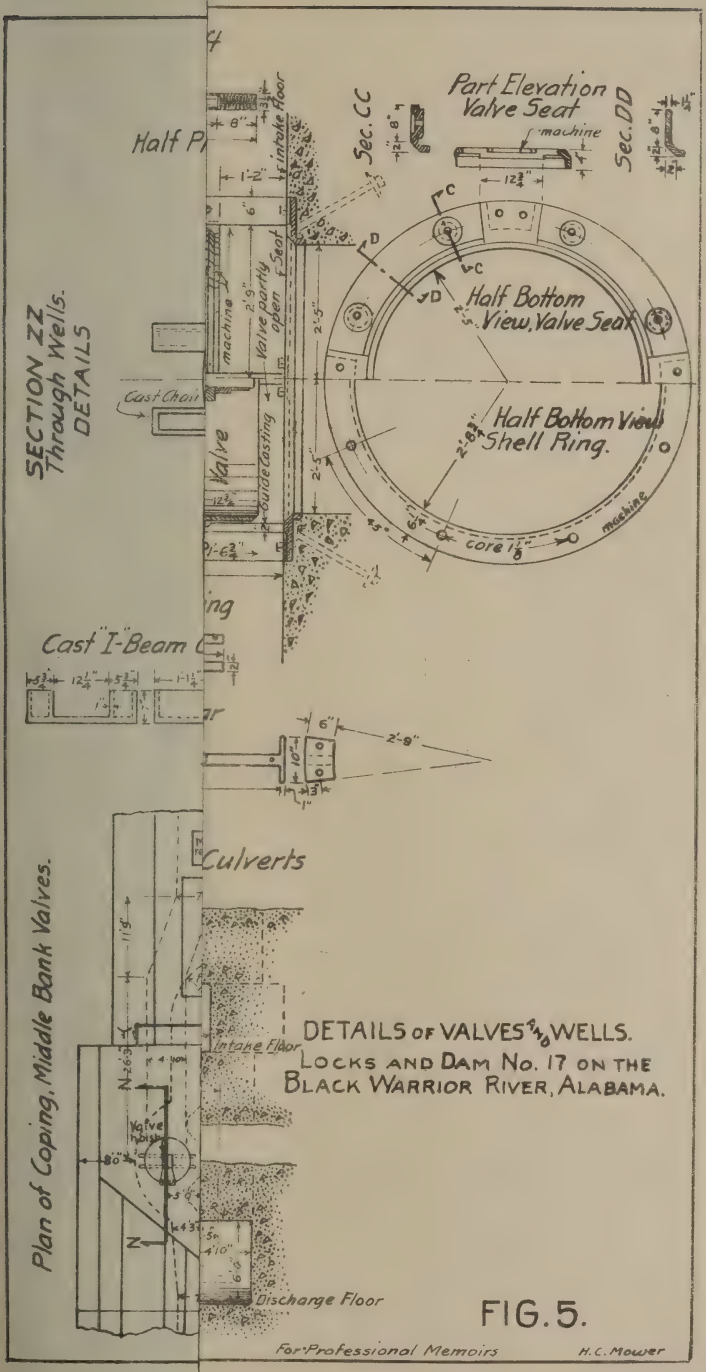
The abutment, which is built 15 feet higher than the crest of dam, connects the end of the dam with the vertical face of rock in the hillside. The plan of the structure shows three sides of a square, the volume enclosed within being backfilled flush with the coping and sodded.

The breakwater wall is 550 feet long and extends downstream from the lower end of the river wall at an angle with the lock axis of 1 on 3. While this wall will contract the width of the river below the dam considerably and probably cause an eddy and currents across the lower approach at the end of the wall, it is believed that boats entering or leaving will have steerage way and be under control at this distance from the lock, and that pilots will have no trouble in coming to rest and making landings under the protection of this wall. The cross wash below the ends of locks now in service where no protection has been provided, makes it difficult and even dangerous, at some stages of river, for boats to enter the locks from below. More especially is this true in approaches where the river becomes narrower below the dams, as is the case at Lock 17.

The retaining wall is built as an extension of the lower bank wall and will serve as a temporary mooring for boats awaiting their turn to lock up. Line hooks, snubbing posts and ladders are built in retaining and breakwater walls as shown on Fig. 4.

The rock bottom in the lower approach was excavated to a minimum depth of 7 feet below pool level for a distance of about 1,000 feet below the lock where the cut ran out. The sides of this cut were channeled as were all vertical rock faces, excepting those to which concrete walls were to be bonded and which were shot off in such a way as to leave rough faces.

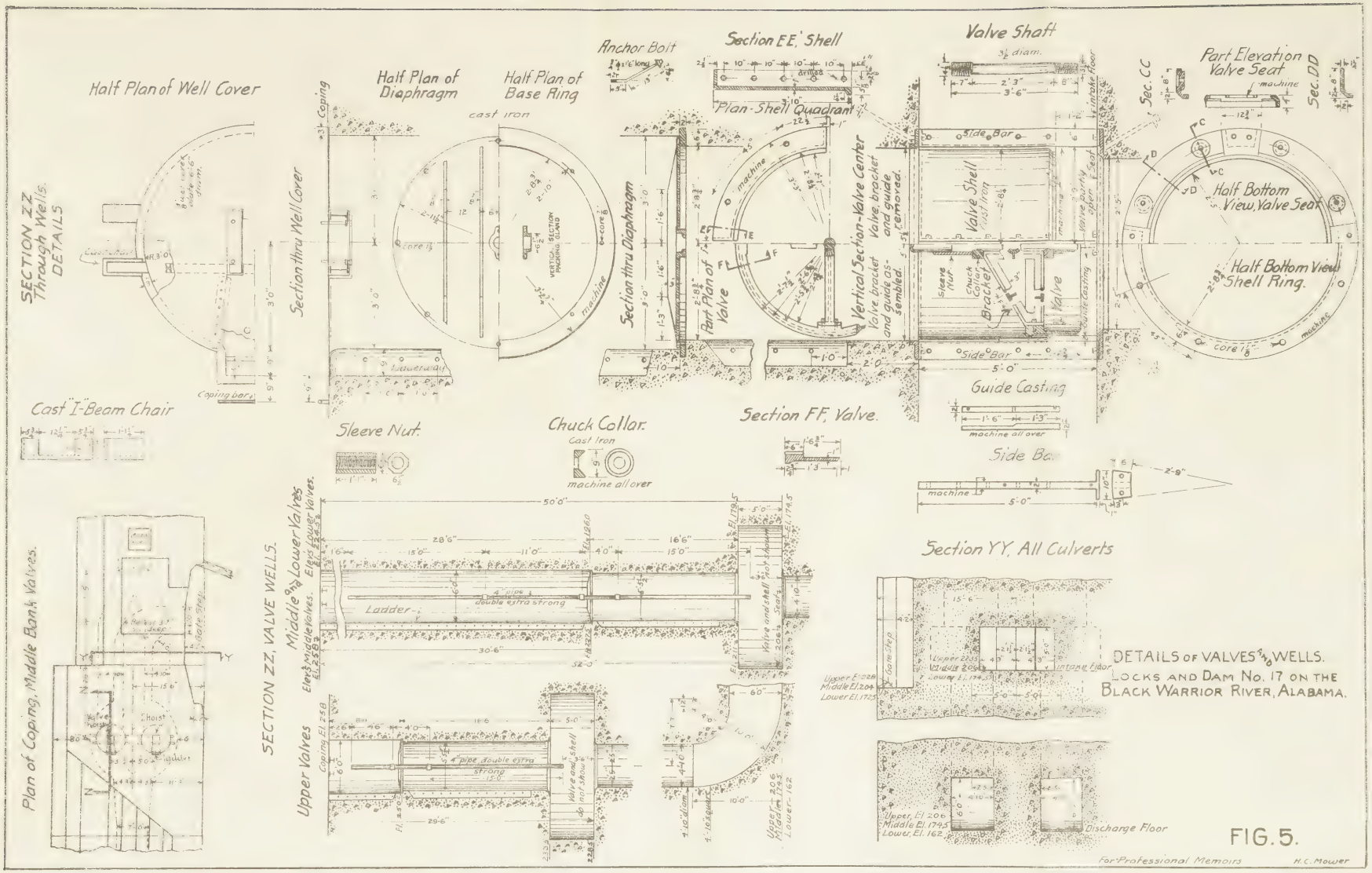
Operating valves at all other locks on the river are of the balanced type, 2 feet 10 inches wide by 6 feet high, swinging on vertical shafts in the culvert intakes as described by Mr. McCalla in his article above mentioned. These valves have been, in most cases, geared together in sets of three. Quadrant sectors keyed on the tops of the three vertical shafts engage with worms on a horizontal shaft which is turned by 22-inch handwheels keyed to the ends. This device works very satisfactorily under 10 and 12 foot heads, but on locks of 14-foot lift it was found that the valves, under full head, could only be partially opened by one operator, and that only after the head had been reduced to about 10 feet could the three valves of a set be opened simultaneously to normal position by one operator. To obviate this delay in filling and emptying the chambers the horizontal shaft of each set of valves was disconnected at the flange joints between two worms, so that the operator could open one valve completely by one hand wheel and the remaining two by using the hand wheel at the opposite end of the horizontal shaft.



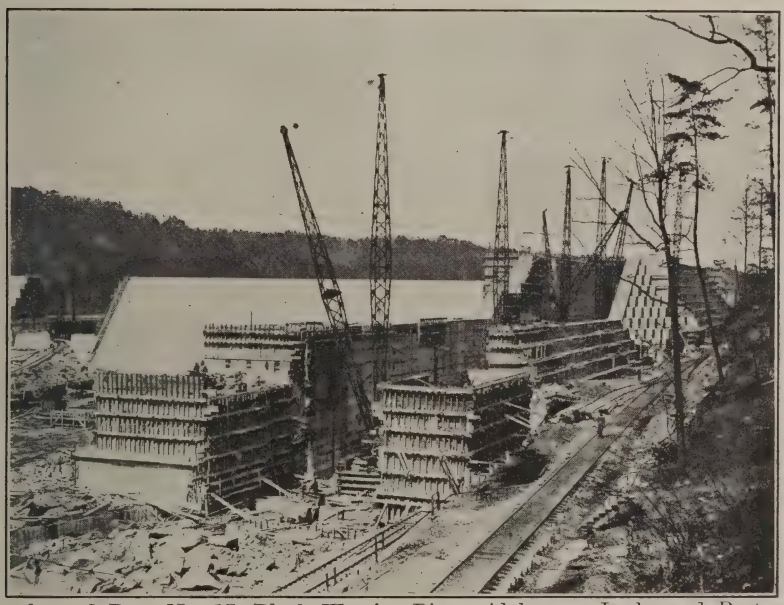
DETAILS OF VALVES & WELLS.
LOCKS AND DAM No. 17 ON THE BLACK WARRIOR RIVER, ALABAMA.

FIG. 5.

For Professional Memoirs H.C. Mower


SECTION ZZ
Through Wells.
DETAILS
Half Plan of Well Cover
Half Plan of Diaphragm
Half Plan of Base Ring
cast iron
Section thru Well Cover
Section thru Diaphragm
Anchor Bolt
Section EE' Shell
Plan-Shell Quadrant
Valve Shaft
Sec. CC
Sec. DD
Part Elevation Valve Seat
machine
Half Bottom View, Valve Seat
Half Bottom View Shell Ring.
Vertical Section-Valve Center
Valve bracket and guide assembled.
Valve bracket and guide assembled. removed.
Valve Shell
Valve
Bracket
Chuck Collar
Sleeve Nut
Guide Casting
Section FF, Valve.
Side Bar
Cast "I"-Beam Chair
Chuck Collar.
Cast Iron
machine all over
Sleeve Nut.
Plan of Coping, Middle Bank Valves.
SECTION ZZ, VALVE WELLS.
Middle & Lower Valves
Elev. Middle Valves. El. 254.5
Elev. Lower Valves. El. 242.5
Upper Valves
Coping El. 258
Ladder
4" pipe double extra strong
Valve and shell not shown
Section YY, All Culverts
Gate Slop
Intake Floor
Discharge Floor
Upper El. 228
Middle El. 204
Lower El. 172.5
Upper, El. 206
Middle El. 179.5
Lower, El. 162
DETAILS OF VALVES & WELLS.
LOCKS AND DAM No. 17 ON THE
BLACK WARRIOR RIVER, ALABAMA.
FIG. 5.
For Professional Memoirs
H.C. Mower

At that time Lock 16 was being built with culverts and valves of the same type to operate under a 21-foot head, and the failure of this device was assured. As a solution of the problem a simple hydraulic laboratory was improvised on an 8-inch suction dredge and a model valve and culvert were carefully built to exactly one-quarter service size. Twenty-three different arrangements of deflection plates were tried, each under seven different heads of water and readings were taken of moments in inch-pounds required to start, open past the quarter, half, three-quarter and normal positions for each arrangement of deflection faces and for



Locks and Dam No. 17, Black Warrior River, Alabama. Locks and Part of Dam from Hillside. April, 1914.

each head. The results were plotted in curves, the arrangement showing the least resistance to operation for all positions of the valve was adopted, and the corresponding change was made on all valves of 14 and 21 foot lift locks. The results have been satisfactory.

The writer does not believe that valves of this type and size will prove satisfactory for hand-power operation under heads exceeding 12 feet, until the experimental adjustment of deflection faces, under local conditions, has minimized the resistance of the discharge to the opening valve for every position of the quarter revolution.

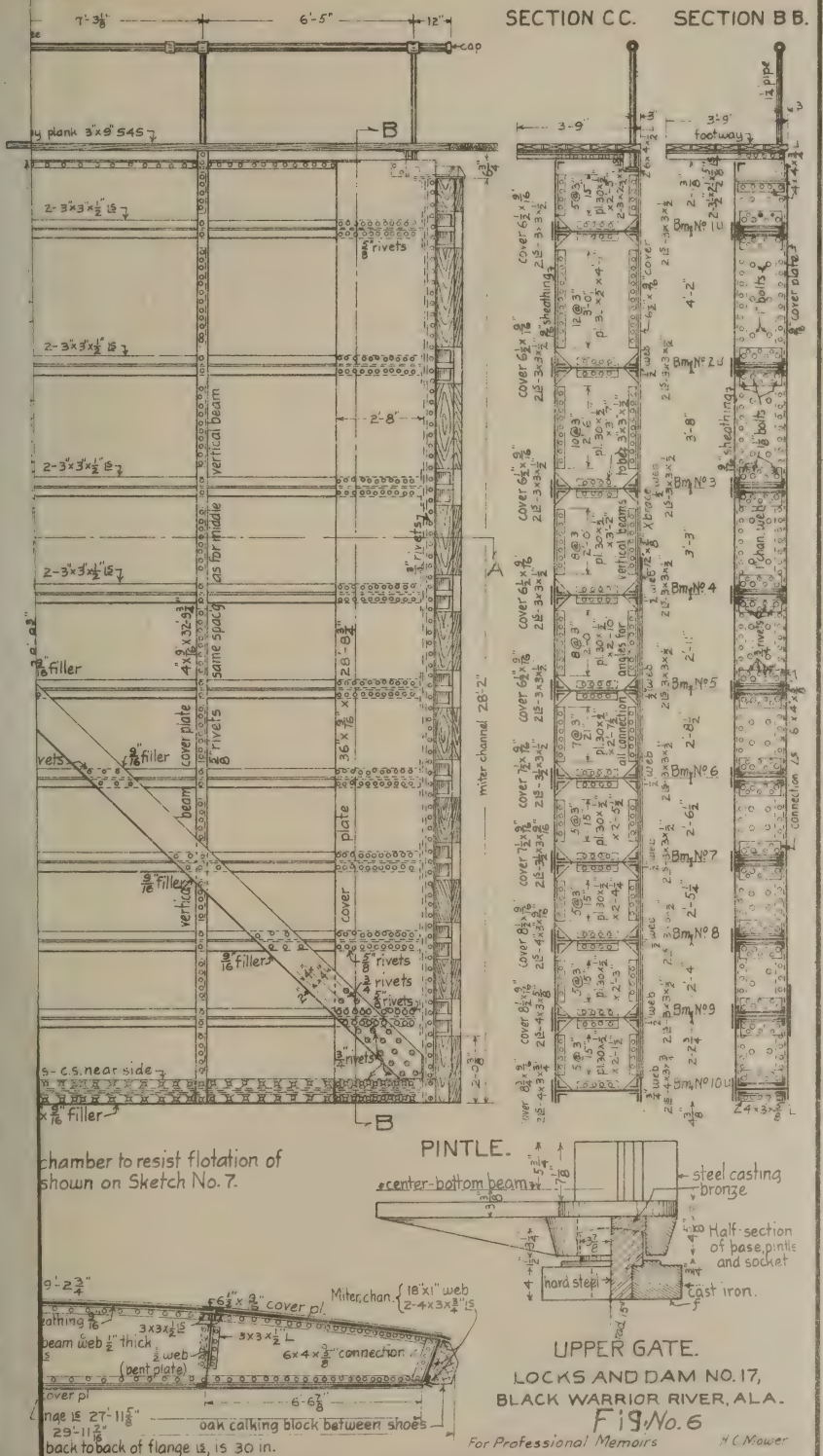
This experience with balanced valves (that will not balance) led to the use of the cylindrical valve at Lock 17, where the normal head is 31.5 feet. The valves, as drawings (Fig. 5) show, are 5 feet 1½ inches in diameter and are lifted 14 inches into cast iron shells. The valve chambers are made 5 feet high to reduce the velocity of approach and the vibration. The drums will be lifted by Coldwell-Wilcox motor-driven hoists, equipped with limit switches installed on the coping over the wells. It will be noted that the valves are set in open wells and that the draft tube vacuum is preserved by cast diaphragms bolted to flange rings over the valve casings or shells. These diaphragms may be lifted out of the wells and the valves inspected periodically for loose cap bolts, all of which are accessible from inside the closed valve when the lock is in commission. In case of the failure of a valve, the diaphragm and valve may be removed from the well, the pools leveled and an emergency diaphragm or cast iron head adjusted over the seat of the injured valve, leaving the lock in commission while necessary repairs are being made. The writer believes the installation in open wells will obviate the only serious objection to the cylindrical valve, that of inaccessibility.

The four valves (two in each wall), opened simultaneously, will fill or drain a chamber in eight minutes and thirty seconds, but the initial rise (or fall) of 10 feet in one minute and fourteen seconds is considered too rapid for safety, and only two valves will be opened under full head, adding open valve area as the head is reduced, keeping the rise (or fall) of water in the chamber within 4 feet per minute.

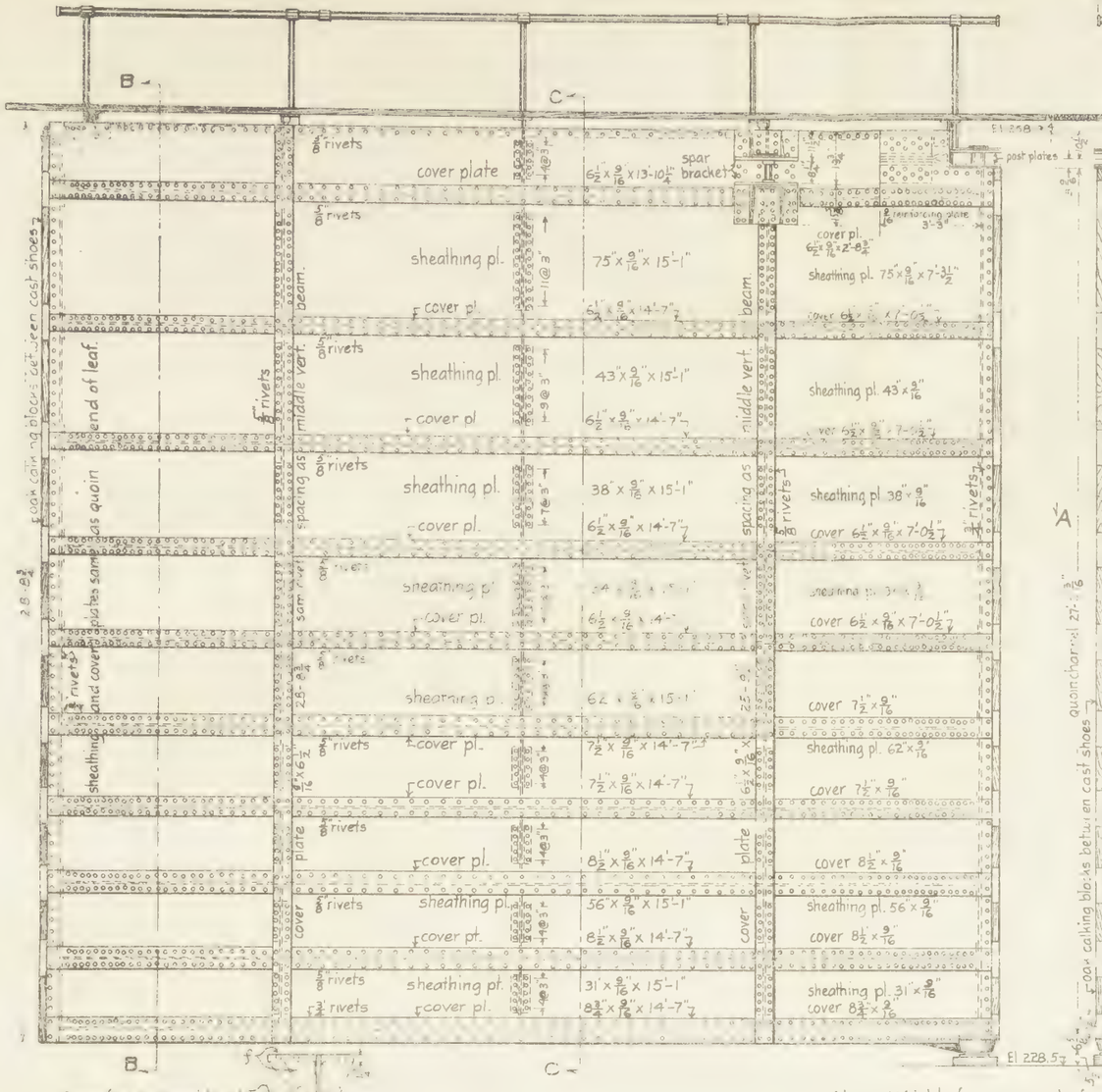
The gates (see Figs. 6 and 7) are of the mitered type, single sheathed, and framed by horizontal built beams with curved backs, to the ends of which are riveted heavy vertical built channels. Into these channels and opposite the ends of beams are set the reaction shoes, on both miter and quoin ends; the space between shoes being filled by oak blocks of the same horizontal section as that of the shoes. The function of these blocks is that of rendering the miter and quoin joints water-tight. A brief outline of the gate design follows:

Allowable unit stresses of twelve, ten and eight thousand pounds were used for tension, compression and shear in all structural steel.

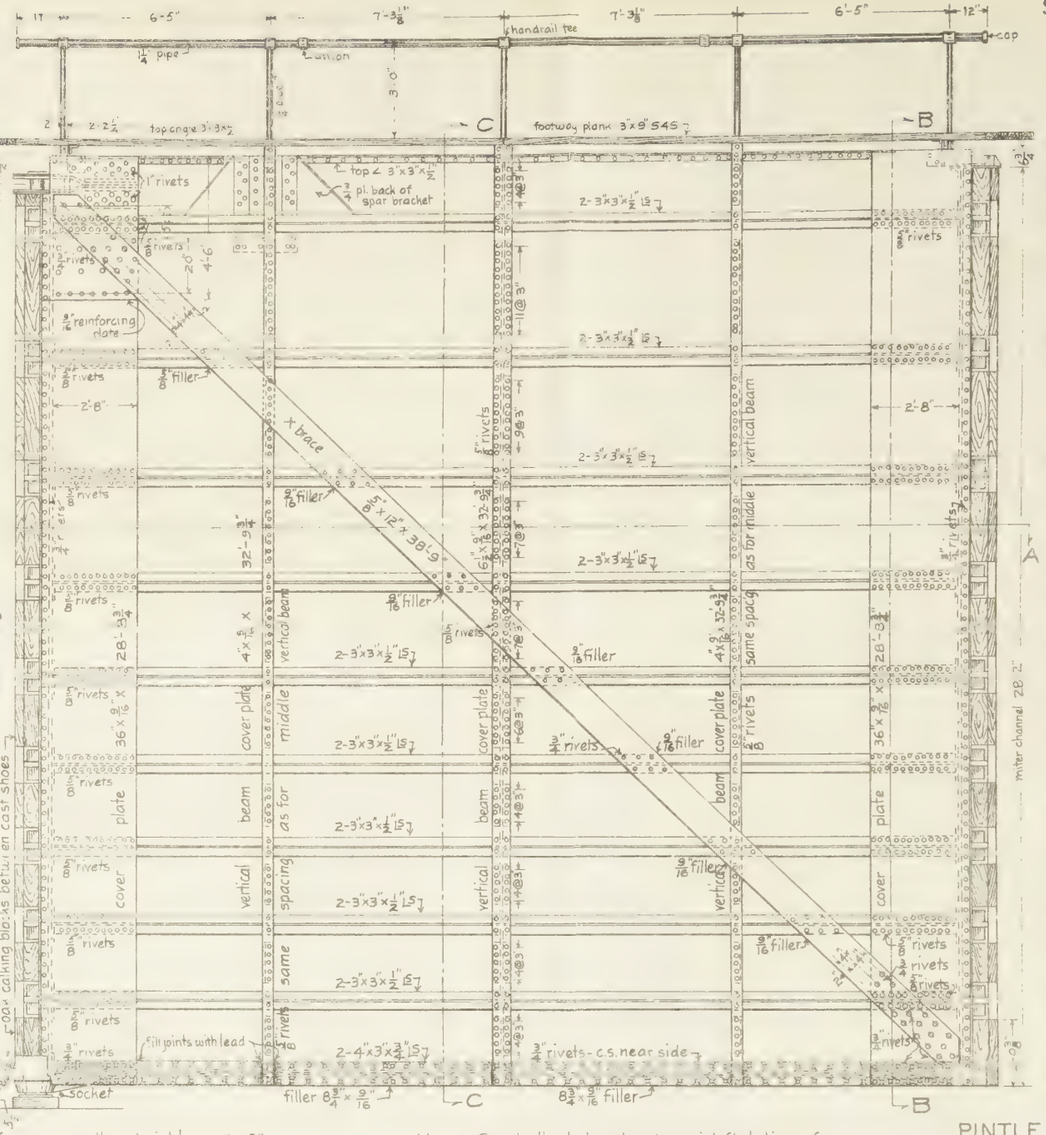
In order to make all parts of the gates accessible for cleaning and painting, 9½ inches was made the minimum clearance be-


VATION.
SECTION C.C.
SECTION B.B.
PINTLE.
UPPER GATE.
LOCKS AND DAM NO. 17,
BLACK WARRIOR RIVER, ALA.
Fig. No. 6
For Professional Memoirs
H.C. Mower
chamber to resist flotation of
shown on Sketch No. 7.
back to back of flange ls, is 30 in.
oak calking block between shoes
steel casting
bronze
Half-section
of pase, pintle
and socket
cast iron.
hard steel
miter channel 28'-2"
cover plate
vertical beam
rivets
filler
footway
sheathing
bolts
connection L5

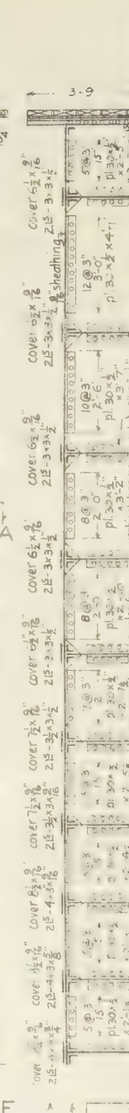
UPSTREAM ELEVATION.



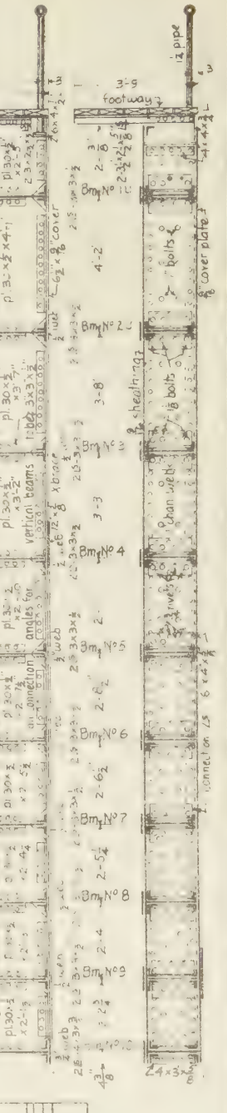
DOWNSTREAM ELEVATION.



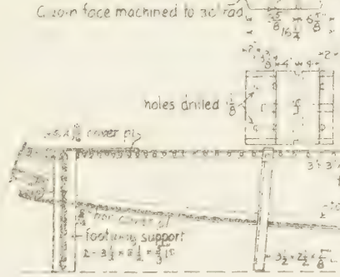
SECTION C C.



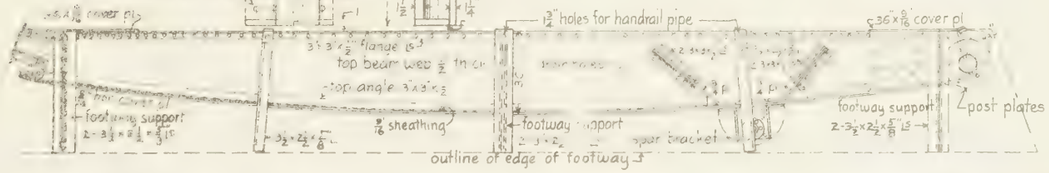
SECTION B B.



CAST IRON SHOE.



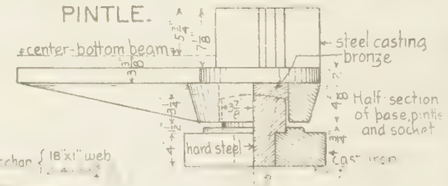
PLAN.



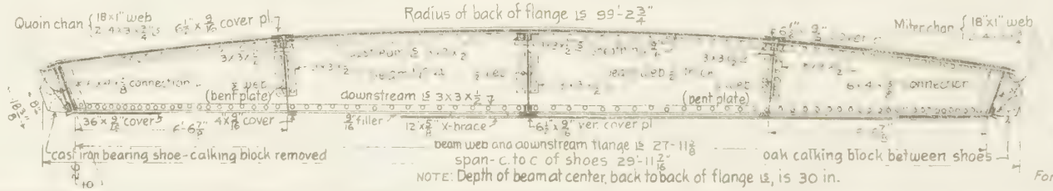
NOTE: Weight of one upper leaf = 86,000 lbs., height = 28'-8¾"
of one middle leaf = 185,700 lbs., height = 52'-10"
of one lower leaf = 177,200 lbs., height = 50'-10"

NOTE: See ballast chamber to resist flotation of upper gates as shown on Sketch No. 7.

PINTLE.



SECTION A A


Quoin chan ∠ 18"x1" web
6½ x ⅜ cover pl.
Radius of back of flange is 99'-2¾"
Miter chan ∠ 18"x1" web
3½ x 3½ x ½ ∠
connection
(bent plate)
downstream is 3 x 3 x ½ ∠
6'-4" connector
36 x ⅜ cover
4'-6½" 4 x 7/16 cover
⅜ filler
12 x ⅝ x-brace
6½ x ½ ver. cover pl
cast iron bearing shoe - calking block removed
beam web and downstream flange is 27'-11⅝"
span - c. to c. of shoes 29'-11½"
oak calking block between shoes

NOTE: Depth of beam at center, back to back of flange is, is 30 in.

UPPER GATE.

LOCAS AND DAM NO. 17,
BLACK WARRIOR RIVER ALA.

Fig. No. 6

For Professional Memoirs H.C. Mower

tween beam flanges on the downstream side, and as the middle gates carry the heaviest load, the bottom and second beams were centered 17⅝ inches. This made the span of sheathing, which is considered a vertical simple beam fixed at ends, 12¾ inches between rivet centers and the load required a depth of 9/16 inch, which was made the uniform thickness of skin for all gates.

Beams were then centered, from bottom to top, so that the spans of sheathing or skin between beams will develop the same fiber stress under the decreasing heads, and the beams were then designed to carry the loads thus distributed.

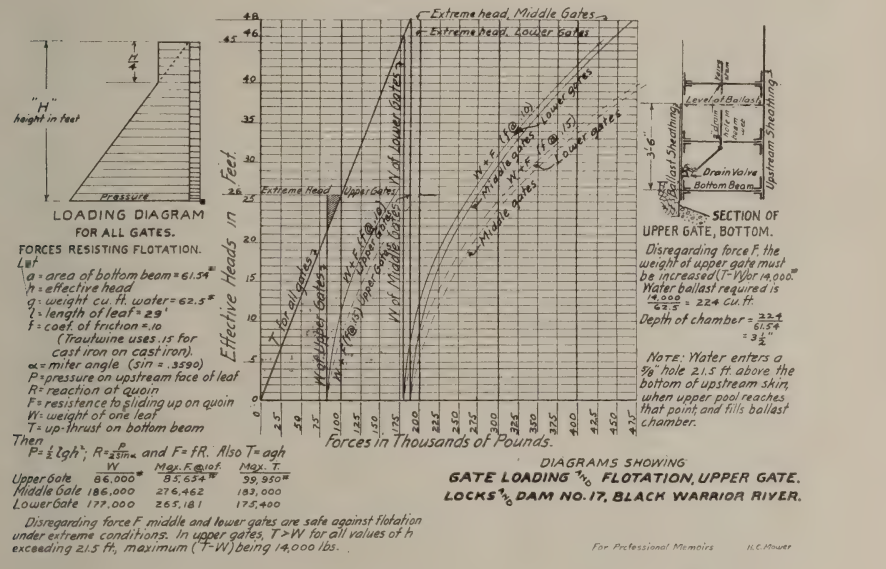

"H" height in feet
Pressure
LOADING DIAGRAM FOR ALL GATES.
FORCES RESISTING FLOTATION.
Let
a = area of bottom beam = 61.54"
h = effective head
q = weight cu. ft. water = 62.5#
l = length of leaf = 29'
f = coef. of friction = .10
(Trautwine uses .15 for cast iron on cast iron).
α = miter angle (sin = .3590)
P = pressure on upstream face of leaf
R = reaction at quoin
F = resistence to sliding up on quoin
W = weight of one leaf
T = up-thrust on bottom beam
Then
P = ½ lgh²; R = P/2sinα and F = fR. Also T = agh
W / Max. F @ 10 f / Max. T
Upper Gate 86,000# 85,634# 99,950#
Middle Gate 186,000 276,462 183,000
Lower Gate 177,000 265,181 175,400
Disregarding force F middle and lower gates are safe against flotation under extreme conditions. In upper gates, T > W for all values of h exceeding 21.5 ft, maximum (T-W) being 14,000 lbs.
Extreme head, Middle Gates
Extreme head, Lower Gates
Extreme Head
Upper Gate
W of Upper Gates
W of Middle Gates
W of Lower Gates
W + F (1 @ 10) Upper Gates
W + F (2 @ 10) Upper Gates
W + F (1 @ 10) Middle Gates
W + F (2 @ 10) Middle Gates
W + F (3 @ 10) Lower gates
W + F (4 @ 10) Lower gates
W + F (5 @ 10) Lower gates
W for all gates
Effective Heads in Feet.
Forces in Thousands of Pounds.
Level of Ballast
Drain Valve
Bottom Beam
Upstream Sheathing
Ballast Sheathing
3'-6"
SECTION OF UPPER GATE, BOTTOM.
Disregarding force F, the weight of upper gate must be increased (T-W) or 14,000#
Water ballast required is 14,000 / 63.5 = 224 cu. ft.
Depth of chamber = 224 / 61.54 = 3½"
NOTE: Water enters a ⅝" hole 21.5 ft. above the bottom of upstream skin, when upper pool reaches that point, and fills ballast chamber.
DIAGRAMS SHOWING GATE LOADING 1/10 FLOTATION, UPPER GATE.
LOCKS 1/10 DAM NO. 17, BLACK WARRIOR RIVER.
For Professional Memoirs
H. C. Mower

Fig. 7.

Beams of all depths between 26 and 36 inches were tried graphically, and the center depth of 30 inches with the back curved to a 99 foot 2¾ inch radius was found to give the most economical distribution of metal. The extreme upstream position of the line of force in this beam, shown in Fig. 6, falls outside the center of gravity of the curved flange at the middle of the beam, showing slight tension in the straight flange on that section. The excessive compression near the ends of the light straight flange is carried by the vertical plates. The beam webs are assumed to carry shear only, with the exception of that portion gripped

between the flange angles, which, with a strip of sheathing the width of the curved flange, is considered part of the flange section. To this end, rivets are spaced to conform to the varying horizontal shear.

Flange stresses were computed and areas determined by formulæ recommended in the Board of Engineers' Report on Deep Waterways, on sections taken every 2 feet, and the results thus obtained for mid-sections were as a rough check compared with the stresses found by treating the beams as eccentrically loaded columns.

The load diagram shown on Fig. 7 was used in the design of all gates, and provides strength, in the beams of the upper quarter, in excess of that required by direct static load. Three vertical beams, built of intercostals between horizontal frames on quarter and mid-sections of each leaf, guard against buckling of horizontal webs and add vertical stiffness to that supplied by miter and quoin post channels and cover plates.

Upper gates are 28 feet 8¾ inches high and weigh 86,000 pounds per leaf; middle gates are 52 feet 10 inches high and weigh 185,700 pounds each, and the lower gates, 50 feet 10 inches high, weigh 177,200 pounds per leaf.

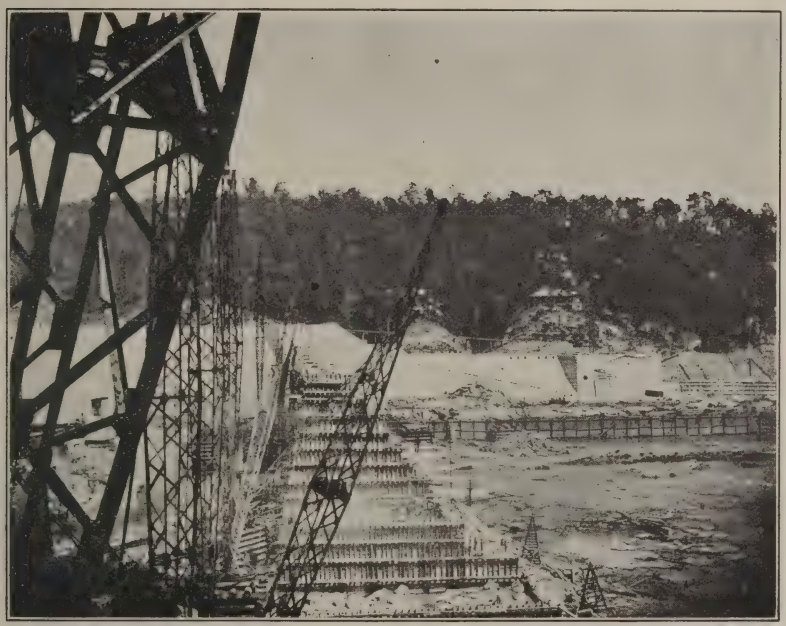
The quoin and miter bearing faces of the reaction shoes are accurately machined to a flat curve to confine the extreme positions of the line of thrust within 2 inches up and down stream from the center.

The cast iron pivot bases are shrunk onto the shanks of steel pintles, the bearing ends of which are turned to convex spherical faces of 15 inches radius. The sockets are heavy steel castings and are bolted to the bottom beams and quoin channels. Pintle bearings in the sockets are bushed with Parson's manganese bronze forgings or "disc metal" tested to 50,000 pounds in compression, and machined to concave spherical faces of 15 inches radius. These bushings are forced into place under 40 tons pressure. One-eighth inch clearance is provided between vertical sides of pintle heads and inside walls of sockets, for while it is expected that the pintles will be centered precisely under the centers of the quoin posts, there is, nevertheless, the possibility of minute displacement, or that of slight lateral crowding when the swinging leaf reaches the mitered position and receives its load. Since the pivot is held rigidly in place, this force would be car-

ried by the pintle head were no clearance allowed between pintle and socket for a slight horizontal displacement.

Ample variation is provided for the adjustment of yoke, pintle base and reaction shoes; the latter to be backed by a special babbitt metal consisting of 20 per cent antimony and 80 per cent lead, which will be poured when the shoes are placed after the gates are erected and have been mitered for the purpose.

Each gate will be operated by a bull wheel and spar device. The crank or bull wheel will make one-half revolution, opening



Locks and Dam No. 17, Black Warrior River, Alabama. Locks and Dam from Abutment Site. June, 1914.

or closing the gate, in 92 seconds and is driven by a 10-horsepower reversible motor through a worm and train of gears. The mechanism is designed to exert a tangential thrust on the spar of 30,000 pounds, above which point the power will be cut-off automatically by the compression of spiral springs in the spar head. These gears are installed in recesses in the copings and are controlled locally.

Up-thrust or flotation was considered while the gates were being designed, and it was decided at that time to recognize the

friction at the quoin as a force acting with the weight of the gate in resisting the up-thrust on the bottom beam. The curves (Fig. 7) show the magnitude of this force and that only in the case of the upper gates does the up-thrust exceed the weight of the gates under extreme conditions. However, as this force has never been recognized in practice, sheathing was placed on the downstream face of the upper gates from the bottom to a height necessary to contain water ballast weighing the difference between extreme flotation and the weight of the gate, as shown on sketch. This chamber is filled automatically when the river reaches the stage where flotation equals the weight of the leaf, and may be drained by a 6-inch flap valve operated from the footway when the river has fallen below the danger stage. Manholes with covers secured by cap bolts render these chambers accessible for cleaning and painting.

The insides of chambers and all surfaces below normal pool levels will be protected by bitumastic enamel, and all steel above water, including gates, maneuvering machinery and special irons, will be painted two coats of red lead and two coats of Detroit Graphite (No. 501 and No. 106).

With the exception of the curved back beams, which were fabricated and delivered by contract, the gates have been built at the government shops under the Tuscaloosa sub-office and will be erected in January, 1915, by the same force. The custom of building and erecting gates by hired labor has been criticised repeatedly, and in some districts the practice would doubtless prove costly and unsatisfactory. The conditions here are unusual, however, in that a small machine shop and force of mechanics have to be maintained throughout the year for keeping in repair the machinery on tow boats, dredges, snagboats, at the boat-yard and for periodically painting and repairing gates, valves and operating machinery on sixteen locks. Seventeen sets of gates have been built and sixteen of these have been erected by the same organization. The workmanship has been satisfactory and costs have compared very favorably with those of contract work of a similar nature in neighboring districts. With the exception of castings, special irons for this lock have also been made at the shops.

Power will be furnished, for the present, by a 30-kilowatt direct current generator which will be direct-connected to a 50-horsepower engine using distillate as fuel. This plant, installed

on the river wall as indicated on Fig. 3, at the portal of the transmission gallery and opposite the main switchboard, will light the houses and lock walls and drive gate and valve operating motors.

Conduits, with pull and outlet boxes, are embedded in concrete walls and connect arc lamps and motors with the main switchboard, one conduit being used for each conductor. Walls will be lighted by arc lamps from the upper end of guide walls to the lower end of the retaining wall. Machinery will be controlled by local rheostats, and in event of failure of any part of power plant or motors, valves and gates may be operated by hand.

Locks and Dam No. 17, Black Warrior River, Alabama. General View of Locks from Hillside. March 22, 1915.

Construction methods and the contractor's equipment present some unique features which will be of interest.

All concrete was mixed in 2-yard lots of 1 : 3 : 6 ratio. "Southern States" cement was used on 28-day tests, except in a few instances where the satisfactory results of 7-day breaks were accepted and cement was used rather than to delay the work awaiting the 28-day breaks. This only occurred after periods during which cement had been used at a rate exceeding 600 barrels per day. Tests were made in the laboratory of the Tuscaloosa sub-office.

The aggregate was crushed sandstone quarried in the lower ap-

proach and in two quarries adjoining the reservation. Sand, with the exception of that used in coping mortar, was rolled from screenings of the aggregate crushers, augmented when necessary by aggregate direct from the bins. River sand was used in mortar for finishing purposes. Concrete of a very thin consistency was used, and into this were placed wetted derrick stones or plums measuring from 1 quarter yard to 4 yards in volume and so distributed that the top of the monolith after a days work was a jagged surface, about half of each plum being embedded and half exposed. The proportion of volume filled by derrick stone varied from 22 to 51 per cent of the total volume, depending on the mass of the monolith.

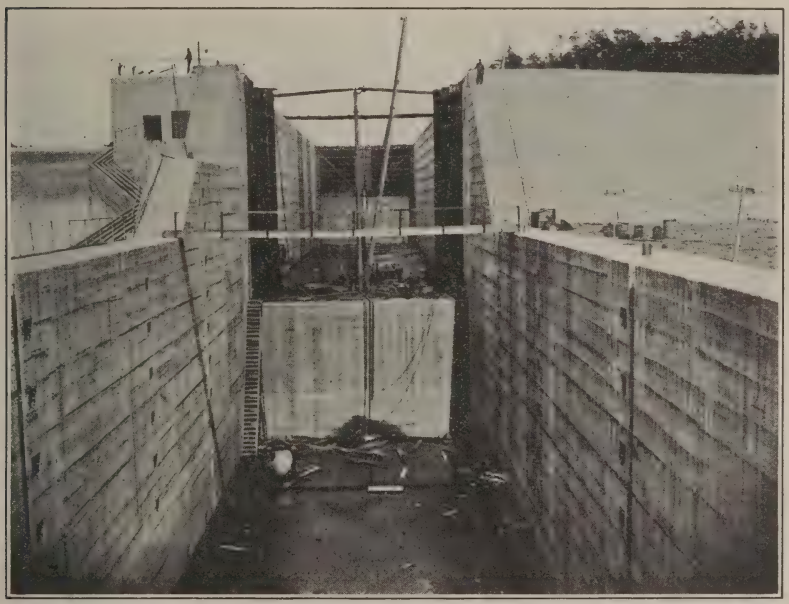
Forms were generally built in panels 5 feet high, which represents the lift of continuous work in the smaller blocks. Each panel is built with a wale at the top through which 7/8-inch tie rods spaced 5 feet center to center pass and are grouted into holes drilled in the tops of derrick stone placed with the previous run of concrete. The bottom of each panel fits into the top of the panel below and is held rigidly to face. This system has been very satisfactory. (See sketch of standard panel, Fig. 4.)

Averaging about 1,000 yards per day, the contractor has confined his concrete work to one shift: 1,328 cubic yards have been placed in one 8-hour shift. Two 8-hour crews have frequently been employed on forming and in the quarries to keep the concrete plant busy. The practice has been to place concrete in three or four monoliths simultaneously.

Pneumatic dump cars, loaded with stone by steam shovels in the quarries are dumped from a trestle into the hopper of a "No. 24K." Allis-Chalmers gyratory crusher. This crusher reduces stone from the size of a cube containing 2 yards to "spall" size and feeds five "No. 5" gyratory crushers in which the spalls are reduced to specification size for aggregate. Automatic conveyors deliver the stone from crusher to crusher, and finally to the storage bins above the mixers. Screenings from all crushers are fed to four pairs of sand rolls, from which the sand is conveyed to the bins. Two extra "No. 5" crushers reduce aggregate, when the screenings fail to supply the required proportion of sand, and the discharge from these crushers passes through the sand rolls above mentioned. Cement is delivered at the mixing hoppers by a belt conveyor which runs the length of the cement house, in which 17,000 barrels are kept stacked in car lots. An additional cement warehouse of 16,000 barrels capacity at Lock 16 is kept stocked.

Two 2-yard "Municipal Cube" mixers, driven by power from the central plant, discharge into 2-yard drop bottom buckets on flat cars and the concrete is delivered on the work in trains of two cars, each carrying four buckets. Concrete and large stone are placed in forms by steel guy derricks of 15 tons capacity, having 115-foot masts and 100-foot booms and so arranged that two or three derricks feed each monolith. No material is dropped or delivered to its bed by the use of chutes.

In the central power plant a battery of eight 150-horsepower



Locks and Dam No. 17, Black Warrior River Alabama. Looking Upstream through Chambers from Footway (Dam shown on left of the picture). March 22, 1915.

internally fired boilers drives one 750-horsepower cross-compound condensing Corliss engine, and three compound Ingersoll-Rand air compressors of 2,400 cubic feet per minute capacity. The engine, which is capable of carrying a 50 per cent overload for eight hours, is belted to the crushers, conveyors and mixers above described, and air from the compressors, at 110 pounds pressure, is piped to the lock site and quarries and drives the planing mill, all derrick hoists, drills and pumps. Steam, outside the central plant, is used only on locomotives, channelers and steam shovels.

A 110-horsepower steam engine drives the 50 K. W. lighting generator which is used for night work.

The contractor built his own line of standard gauge track from Searles, Ala., to the reservation, a distance of 15 miles, over a rough range of hills. The maximum grade is 5 per cent, and two switch-backs were used on the location. Over this road all plant, coal, and other supplies have been hauled by Shay and Heissler geared locomotives.

The contractor was delayed for some weeks during the recent discussion over the Rivers and Harbors bill, and was permitted to

Locks and Dam No. 17, Black Warrior River, Alabama. Lower End of Locks and Dam from End of Breakwater. March 22, 1915.

resume contract work on the morning of October 6, 1914. His estimate for the balance of October amounted to $213,970.69, which is the largest monthly estimate rendered on the contract.

At this writing all concrete has been placed and with the exception of a little backfilling and riprap, the contract work has been finished.

Before leaving this subject the writer wishes to say a word which, since he has had no part in the actual construction, he can do with all propriety. The alignments of walls, true unmarred concrete faces, the precise location of valves and special irons, and the general high standard of workmanship which marks the

completed work, attest to the successful methods employed by the contractor as well as to the efficient work of the engineers and inspectors who have represented the United States in the field.

Water Power. Much more space has already been devoted to this paper than the writer had expected to use, although the treatment of the subject has been most superficial. No attempt will here be made to go into the questions of flow, precipitation, evaporation, etc., but since water power development was contemplated in changing the lift of the dam at Lock 17, the possibilities in that direction seem to demand mention.

Locks and Dam No. 17, Black Warrior River, Alabama. Looking Upstream into Lower Approach, March 22, 1915.

Plans for developing hydro-electric power at this site will, in all probability, be begun as soon as the question of water rights is disposed of. One company has made an exhaustive study of the project, and the program recommended by its engineers is, briefly, as follows:

The mean low water discharge at this point is about 140 second feet, while the maximum flow is about 140,000 second feet. The minimum discharge is to be raised to 2,000 second feet by impounding the water of the flood season at Sanders Ferry on the Mulberry Fork, 56 miles above Lock 17, and releasing the water from the reservoir through power turbines at the rate of 2,000 sec. ft.

The crest of the spillway at the regulating dam is shown on tentative plans at elevation 350.0, being 106 feet above normal pool level, and a draw-off of 30 feet will have a volume of 880,800 acre feet. About 17,500 electric horsepower may be developed by the generators under this plan, 13,000 at the power plant on Lock 17 dam, and, if desired, about 4,500 may be generated at Lock 16. It is probable that an auxiliary steam plant would be needed in exceptionally dry years to maintain the output when the draw-off above the controlling dam exceeds 30 feet.

Traffic. Locks 2 and 3 of the system have been completed only within the past two months* and prior to this time shipping on the upper river had been cut off from Mobile during the low-water months. Although scattered local traffic amounting to a very light tonnage, as shown by the Chief of Engineer's Reports, has followed the completion of the different locks, no attempt has been made to run regular lines of through boats on account of the hazards in passing the unimproved reach of river below Demopolis and danger of being caught above that point by a falling river. Since the completion of Locks 2 and 3, six steel self-propelling barges, each about one thousand tons capacity, are regularly engaged in coal traffic between Gilmore, Ala. (above Lock 13) and New Orleans, and the second of a fleet of coal barges which is being built by one of the largest coal interests with holdings above Lock 17, has been launched at their shipyards above Tuscaloosa.

Three different attempts have been made to locate a locked canal into Birmingham from the Lock 17 pool, but owing to the excessive rise, which is about 340 feet in the length of 55 miles, and the prohibitive cost of feeding the canal with artificial flumes from sources north and northeast of the city, the project has been at least temporarily abandoned. People of this locality are enthusiastic over the benefit to be expected from the completed improvement, and private interests are now discussing the practicability of constructing a double-track road from Birmingham, through Bessemer and thence to the Locust Fork of the Black Warrior.

It is impossible to say at this time how rapidly shipping activity will develop, but indications are that the tonnage which will be transported by this river three years hence will have demonstrated the feasibility of improving and utilizing our inland waterways in sections where undeveloped resources await reasonable transportation.

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*This article was written in December, 1914.—Ed.

The Engineers and the Mexican War.

BY

Maj. WILDURR WILLING,
Corps of Engineers.

All told there were forty-four officers of engineers, including the Chief Engineer of the Army himself, who participated in the war with Mexico in 1846-1847. Of these, nineteen were of the Corps of Engineers and twenty-five of the Corps of Topographical Engineers. Of the former all but one, and of the latter all but four, were graduates of the United States Military Academy, in classes ranging from 1805 to 1846. The total number of officers of each corps as shown by the Army Register of 1848 was: Corps of Engineers, 43; Topographical Engineers, 36.

From the roster of participants we may select from the Corps of Engineers such names as Joseph G. Totten, John G. Barnard, Robert E. Lee, Joseph K. F. Mansfield, P. G. T. Beauregard, Henry W. Benham, Henry W. Halleck, Isaac I. Stevens, George B. McClellan, and Gustavus W. Smith; and from the Corps of Topographical Engineers such names as Joseph E. Johnston, William H. Emory, William B. Franklin, George G. Meade, John Pope, and Thomas J. Wood.

It is not the purpose of the present article to dogmatize upon the capacity of engineer officers to command troops, nor to attempt to select from the listed names the foremost soldier of the American Continent, nor yet to dwell upon past glories in the interest of the present.

The Mexican War was the most successful ever waged by the United States Army; indeed, in our military history it stands out in splendid isolation. The young engineer officers played a part in it that must ever be a source of inspiration and of profitable study to their successors in that branch of the service. It is more for the purpose of refreshing our memories as to the nature of their duties and the manner of their performance of them that these lines are attempted.

General Taylor's long period of waiting on the Nueces and Rio Grande having terminated in the victories of Palo Alto and Resaca de la Palma on Texas soil, the pursuit of the Mexican army was pushed to the Rio Grande; but here it was brought to a sudden halt. As Gen. Cadmus Wilcox remarks: "The full fruits of the victories of the 8th and 9th of May (1846) were not secured by General Taylor, owing to his want of means for prompt pursuit. In active military operations a well organized ponton train is indispensable and, whilst encamped at Corpus Christi, he had not failed to call the attention of the War Department to this subject. Had this want been supplied as requested, he would doubtless have captured all the public property in Matamoras, or the enemy would have been compelled to destroy it, and in the demoralized condition of the Mexican forces a great number of prisoners must have been captured." General Taylor's exact words are to be found in his letter to the Adjutant General from Corpus Christi dated August 26, 1845: "A moderate supply of pontons and ponton wagons might greatly facilitate any active operations in this country, where it is next to impossible to bridge the streams, owing to the scarcity of timber."

The report of Captain Thornton's disastrous contact with the invading Mexican cavalry under Torrejon on April 25, 1846, having reached Washington on May 9, the very day on which the main Mexican force was defeated at Resaca and driven back upon the river, Congress at last took up the question of increasing the regular army and of raising a force of volunteers to prosecute the war. The law authorizing a company of regular engineer soldiers was signed by the President on May 15 and to Capt. Alexander J. Swift, who had recently taken a special course in military engineering at the French engineer school at Metz, was assigned the task of recruiting and conducting it to the frontier where it arrived at Camargo on November 2.

The duties performed by the engineer officers in the Mexican War may be broadly classified under four heads, viz: (a) General Staff, (b) Technical Staff, (c) Aides, (d) Pioneers. Perhaps the most important were those of the first class, in particular those pertaining to what is known in our present Field Service Regulations as the intelligence section.

Either General Scott or General Taylor might with equal propriety have been the author of Frederick's celebrated saying to the effect that if we had accurate information of the enemy we

should beat him every time. It is true that before and during the period of the Mexican War we had no general staff in its present accepted sense. General information of Mexico was so ridiculously lacking that the Quartermaster General could not tell whether wheeled transportation would be suitable in case of invasion. But when our forces entered the theater of war both Generals Scott and Taylor showed themselves past masters in the art of gaining intelligence in the presence of the enemy. The young engineer officers were utilized to the fullest extent of physical endurance on this work.

While yet at Corpus Christi, General Taylor had thorough reconnaissances made by the engineers of possible camp sites in that vicinity with a view to the possibility of a change during the winter to be nearer a more abundant supply of wood. In addition, the entire country between the Nueces and Rio Grande was examined in case this territory should become a theater of war through a Mexican invasion north of the latter stream. The engineers were also utilized to make numerous "preliminary surveys" of the lagunas and streams along the Texas coast.

At his repeated and earnest solicitations General Taylor was authorized by the President on January 13, 1846, to move forward to the Rio Grande. He ordered a special reconnaissance to determine whether the Padre Island or the mainland route were the more practicable for the movement contemplated.

General Taylor, marching by the mainland, left Corpus Christi on March 8 and reached Point Isabel on March 25 with the cavalry where he reported: "The engineer officers are now examining the ground with view to tracing lines of defense and strengthening the position" (selected as the base).

On March 28 the camp opposite Matamoras was reached by the United States forces and steps to fortify it immediately taken. This work was done under the direction of Captain Mansfield, General Taylor's chief engineer. On this subject the commanding general reported to Washington: "On our side a battery for four 18-pounders will be completed, and the guns placed in battery, to-day. These guns bear directly upon the public square of Matamoras, and within good range for demolishing the town. Their object can not be mistaken by the enemy, and will, I think, effectually restrain him from any enterprises upon our side of the river. A strong bastioned field fort, for a garrison of 500 men, has been laid out by the engineers in rear of the battery, and will

be commenced immediately. This work will enable a brigade to maintain this position against any Mexican odds, and will leave me free to dispose of the other corps as considerations of health and convenience may render desirable." Captain Mansfield remained in charge of the construction of the work and during the attack was conspicuous for valor, being within "six or eight feet" of Major Brown when he received his fatal wound.

In his report of the battle of Palo Alto General Taylor remarked, "While the columns were advancing, Lieutenant Blake, topographical engineers, volunteered a reconnaissance of the enemy's line, which was handsomely performed, and resulted in the discovery of at least two batteries of artillery in the intervals of their cavalry and infantry."

Poor Blake, the next day at Resaca de la Palma, accidently killed himself with his own pistol.

Of the latter engagement we read in the commanding general's report: "Lieutenant Scarritt, engineers, and Lieutenants Blake and Meade, topographical engineers, promptly conveyed my orders to every part of the field," thus performing, as it were, the duties of aides.

The forces of the United States made a belated crossing of the Rio Grande; in possession of that stream, the movement on Monterey via Camargo as a secondary base was adopted. In connection with this plan Captain Sanders, of the Corps of Engineers, was ordered to New Orleans to furnish data to the Quartermaster Department as to the nature of the vessels that would be required on the water portion of the line of communications.

The army of invasion encamped before Monterey on September 19, of which place General Taylor reported: "It was immediately discovered that the enemy occupied the town in force, and had added greatly to its strength by fortifying the approaches and commanding heights. A close reconnaissance was made the same evening by the officers of engineers and topographical engineers, on both flanks of the town, and it was determined, from the information procured, to occupy the Saltillo road in rear of the town, carrying, if practicable, the several fortified eminences in that direction." The reconnaissance of the Saltillo road was made by Major Mansfield (brevetted major for gallant conduct at Fort Brown) and of the eastern approaches by Captain Williams. The examination of the Saltillo road by Mansfield proved the entire practicability of seizing this road west of the town.

Upon the information furnished by the engineers was based the commanding general's decision, which was to send General Worth to make with his division a wide turning movement to the west around the bishop's palace at the extreme left of the Mexican line while Generals Twiggs and Butler were to occupy the enemy's right flank and front. To Worth's column were assigned Capt. John Sanders and Lieut. George Gordon Meade of the engineers.

In the march across country to the westward on the afternoon of September 20, Captain Sanders was given the task of making the road practicable for artillery and the progress was slower than General Worth had anticipated. The Saltillo road was occupied on the 21st and the strong positions on Federacion and Soldado hills captured. The assault on Independencia, the main work on the western front of the city was begun at 2 a. m. on the 22d. The column of attack was conducted on its successful mission to the point of attack by Sanders and Meade. The latter made a topographical sketch of Worth's movements and together with the former was commended "to special consideration" in the general's report. He spoke of Meade's "intelligent zeal and gallantry."

The assault on the eastern front of the city proved a bloody affair. Garland's column of attack was accompanied by Major Mansfield, Captain Williams, and Lieutenant Pope, "Major Mansfield being charged with its direction, and the designation of points of attack." The Mexican battery No. 1 was captured on this date after Butler and Twiggs had come to Garland's assistance, the operations on this front being under the direction of General Taylor himself. "Intrenching tools were procured, and additional strength was given to the works and protection to the men, by working parties during the night, under the direction of Lieutenant Scarritt, engineers."

By night of September 23, Quitman from the east and Worth from the west had entered the city and advanced "from house to house, and from square to square" toward the plaza. During the night Quitman was withdrawn to the outer works. On the 24th the Mexicans displayed the white flag.

General Taylor, in his official report of the engagements around Monterey, states: "I must express my particular obligations to Brevet Major Mansfield and Lieutenant Scarritt, Corps of Engi-

neers; they both rendered most important services in reconnoitering the enemy's positions, conducting troops in attack, and strengthening the works captured from the enemy. Major Mansfield, though wounded on the 21st, remained on duty during that and the following day until confined, by his wound, to camp. Captain Williams, topographical engineers, to my great regret and the loss of the service, was mortally wounded while fearlessly exposing himself in the attack of the 21st. Lieutenant Pope of the same corps, was active and zealous throughout the operations."

On the 13th of November, General Taylor's truce having been terminated by direction of the President, he started Worth's division for Saltillo, where it arrived without opposition. Near here General Wool joined in December with his column, which he had marched from San Antonio via Monclova and Parras. In General Taylor's absence at Victoria, General Wool assumed command of the combined forces around Saltillo. General Taylor resumed command on February 2, 1847, and at this time his foremost columns were at Agua Nueva, 18 miles south of Saltillo, where they remained until the night of February 21-22.

The combined engineers of Taylor and Wool now made daily reconnaissances of the country in the vicinity of Agua Nueva, and General Benham tells us that they "Soon developed the fact that these mountains were passable in every direction, and that the camps in the valley were in a most dangerous position. . . . These reports that we made soon gave an earnest anxiety to our usually imperturbable general." General Taylor in his official report of the battle states: "As the camp at Agua Nueva could be turned on either flank, and as the enemy's force was greatly superior to our own, particularly in the arm of cavalry, I determined, after much consideration, to take up a position about 11 miles in rear and there await the attack."

The foregoing movement was not determined upon, however, until the enemy in great strength was reported at Encarnacion 30 miles in his front, by Major McCullough, in charge of the party of Texas spies, and by Lieutenant Colonel May of the cavalry. So slow was General Taylor to act upon his information and so rapid was the advance of the Mexican column that the commanding general was indeed surprised at Buena Vista before his retrograde movement had been completed. He had ordered Mansfield and Benham, under escort of Rucker's squadron of

dragoons, to make a reconnaissance south of the old camp at Agua Nueva toward Encarnacion on the morning of February 22. These officers had scarcely gotten out of camp at Buena Vista when the enemy were reported in great force only a short distance away.

In the engagement at Buena Vista Lieutenant Bryan, of the Topographical Engineers, served with Washington's battery of artillery, where he was slightly wounded. Lieutenant Sitgreaves was "conspicuously noticed by the general for gallantry and good conduct." Major Mansfield and Lieutenant Benham of the Corps of Engineers, and Captain Linnard and Lieutenants Pope and Franklin of the Topographical Engineers were engaged in reconnaissance and in carrying the general's orders to various parts of the field. Lieutenants Benham and Sitgreaves were especially mentioned in General Wool's report.

While General Wool's expedition with the "Centre Division" against Chihuahua failed of its original purpose and was finally diverted to Saltillo in time to reinforce General Taylor at the decisive battle of Buena Vista, its early movements are not without interest to the engineer.

The column was based on Port Lavaca in Matagorda Bay with San Antonio as an intermediate base. Capt. Robert E. Lee was its chief engineer. The other engineer officers who accompanied it were: Capt. George W. Hughes, Capt. William D. Fraser, and Lieuts. Lorenzo Sitgreaves, William B. Franklin and Francis T. Bryan.

From the sea Hughes and Sitgreaves followed the Goliad road, while Franklin and Bryan took the the Gonzales road to San Antonio for the purpose of making accurate topographical surveys of these two routes. The survey parties were equipped with accurate instruments and chronometers for determining latitude and longitude.

The engineers left San Antonio on September 23, 1846, followed on the 28th by Wool's main body. Hughes and Sitgreaves did the reconnaissance while Franklin took the observations. Captain Hughes in his report tells us: "Even when the army was at a halt, the topographical engineers were busily engaged reconnoitering the country for considerable distances, sometimes 80 miles from camp."

A few extracts from the official reports will indicate the nature of the work performed, "The road to Lucky's creek is good. We found the creek, contrary to expectation, full of good water, 2 feet deep, and flowing clearly and rapidly. On the west

bank is a lovely grove of large live oak, and a rich valley, almost boundless to the eye, of excellent grazing. An army of 10,000 cavalry might encamp here for a week, and not exhaust its luxuriance. . . . Accompanied by Lieutenant Franklin, made a reconnaissance of the river (Leona) for 7 miles above the camp on the east bank, searching for another ford, which proved unsuccessful. For the whole of that distance, and apparently for a much greater distance, the valley is wide, well timbered, and extremely rich. . . . Five miles to the Saladino, a very pretty, small stream of pure water, which has been always represented as being brackish. The natural crossing was deep and boggy; but a bridge erected by Colonel Harney having been repaired by the engineers, we were enabled to cross without difficulty, after a short delay."

The engineers reached the Rio Grande, 164 miles, on October 8. Captain Fraser had constructed some pontons in San Antonio. On October 11, a flying bridge was put into operation and Wool's force of about 2,000 men was ferried across on the 12th, together with all the trains. The river was about 800 feet wide. At the crossing defensive works were constructed under the supervision of the engineers, and two companies of volunteers left behind as a garrison.

The pontons were left behind on the Rio Grande and the army had several disagreeable experiences in crossing streams. Heavily loaded wagons were utilized as points of support and bridges constructed for the passage of the infantry.

General Wool reached Monclova on the 30th of October and remained there until the 23d of November. "During this long halt the topographical engineers were engaged in making surveys of the surrounding country, and astronomical observations, reconnaissances for long distances from the camp in different directions, drawing maps, and reducing the previous observations." Throughout the march Captain Hughes was assigned the task of selecting and laying out camps and of assigning the troops to their ground. He habitually preceded the advance guard by several hours. From Monclova the column proceeded to Parras, from which place it marched to Agua Nueva, covering the last 100 miles in four days. There was some daring reconnaissance work done by the engineers while Wool and Taylor were yet separated. Franklin, on the 21st of November, was sent to Mon-

terey with despatches for the latter, making the round trip in six days and doing reconnaissance work both ways.

Of the peregrinations of Bvt. Capt. John C. Frémont, Corps of Topographical Engineers, in California, both before and after hostilities with Mexico began, there is not time to speak. Those interested in the "Pathfinder" and in early California history will find the official reports of much interest.

It is necessary now to give a brief sketch of another column which was organized with Fort Leavenworth as a base, and known as the "Army of the West." It was commanded by Col. Stephen W. Kearney, First Dragoons. As the troops arrived at the base they were sent forward to Bent's Fort, where the old Santa Fé trail struck the Arkansas River. On August 1, 1846, the command of Colonel Kearney, about 1,800 strong, marched from Bent's Fort for Santa Fé, the capital of New Mexico.

The engineers with this expedition were organized into a party similar to that which accompanied General Wool. The personnel consisted of First Lieutenants W. H. Emory and W. H. Warner and Second Lieutenants J. W. Abert and G. W. Peck, of the topographical engineers. Lieut. Jeremy F. Gilmer, Corps of Engineers, also accompanied this expedition. The notes and route sketch of the march to California were published as a Senate document. Abert through illness was left behind at the Arkansas, and Peck at Santa Fé. Warner was subsequently killed by Indians in the Sierra Nevada Mountains in 1849.

Somewhat amusing is Emory's entry of August 13, 1846: "as soon as we commenced the descent into the valley of Moro creek, someone reported a company of Mexicans at the crossing; Colonel Kearney ordered me to go forward with twelve of the Laclede rangers, and reconnoiter the party, and if they attempted to run, to pursue and capture as many as we could. As Lieutenant Elliott and myself approached this company, they appeared to be motionless, and on coming up, we found them to consist of nothing but the pine stakes of a corral."

A few extracts from the report may prove of interest and indicate the nature of the work. "Fifteen miles from Santa Fé we reached the position deserted by Armijo. The topographical sketch, by Lieutenant Peck, will give some idea of it. It is a gateway which, in the hands of a skilful engineer and one hundred resolute men, would have been perfectly impregnable. . . . Armijo's arrangements for defense were very stupid. His abattis

was placed behind the gorge some 100 yards, by which he evidently intended that the gorge should be passed before his fire was opened. This done and his batteries would have been carried without difficulty. . . . I received an order to make a reconnaissance of the town (Santa Fé) and select a site for a fort, in coöperation with Lieutenant Gilmer of the engineers. This occupied me diligently on the 19th and 20th and on the 21st the general was furnished with the map. . . . The site selected and marked on the map is within 600 yards of the heart of the town, and is 60 to 100 feet above it. The contour of the grounds is unfavorable for the trace of a regular work, but being the only point which commands the entire town, and which is itself commanded by no other, we did not hesitate to recommend it. The recommendation was approved. On the 22d we submitted a complete plan of the work, which was also approved. It is computed for a garrison of 280 men. On the 23d, the work was commenced with a small force; on the 27th, 100 laborers were set to work on it, detailed from the army; and, on the 31st, 20 Mexican masons were ordered. . . .

"The road from Santa Fé to Ft. Leavenworth presents few obstacles for a railway, and if it continues good to the Pacific, will be one of the routes to be considered, over which the United States will pass immense quantities of merchandise into what may become, in time, the rich and populous states of Sonora, Durango and Southern California."

Lieutenant Emory made some very valuable studies of the various Indian tribes encountered and of the topography of the upper Rio Grande, Gila and Colorado rivers, en route to California.

The engineers were present at the battles of San Pasqual and Los Angeles, and made sketches of the battlefields. In the former engagement Lieutenant Warner was badly wounded, receiving three wounds.

At Los Angeles a fort was constructed for 100 men. Emory tells us: "It was therefore desirable to establish a fort, which, in case of trouble, should enable a small garrison to hold out till aid might come from San Diego, San Francisco, or Monterey, places which are destined to become centers of American settlements."

All told, Kearney's column marched over 2,000 miles.

The first service performed by the engineer company was on General Patterson's march from Matamoras to Victoria between

December 21, 1846, and January 4, 1847. The "pick and shovel" brigade (so dubbed by the line) had the job of rendering the road practicable for the trains, which was successfully done.

One day Major McCall of the General's staff rode up to Lieut. Gustavus W. Smith, in command of the company, and announced that there was a stream ahead which he feared would delay the column for two days. Upon reaching the stream and estimating the situation, Smith amazed the general and his staff by asking for a detail of 800 men as a working party. This detail was furnished and the wagons were crossing about three hours after work began. The distance covered to Victoria was 234 miles.

On January 13 the engineer company, aided by two line companies, left Victoria for Tampico with instructions to continue the work of pioneers so well performed hitherto. General Twiggs was in command of the column which arrived at Tampico after a march of 120 miles on the 23d. Gustavus W. Smith and George B. McClellan, both of the Corps of Engineers, were the officers assigned to the company. McClellan was 20 years old.

Tampico had been occupied by the Navy on November 14, 1846. It had been fortified and garrisoned by the Mexicans, but was abandoned under orders from Santa Anna in October. Captain John G. Barnard and Lieutenant P. G. T. Beauregard were assigned the duty of revising the system of fortifications.

We now come to General Scott's expedition from Vera Cruz as a base in which the engineers played such an important and brilliant part. The transports sailed from Lobos Island on March 2d and arrived off Anton Lizardo, 18 miles southeast of the city, on the 7th. The commanding general early gave evidence of the complete confidence he placed in the engineers by taking a number of them with him on the *Petrita* when he made his rapid survey of the coast above and below the city with a view to selecting a landing place. The troops were landed near Sacrificios on March 9. Worth's division was the first ashore and with it was the engineer company. These troops formed the right of the American line and were in position by the night of the 9th. On the 10th General Patterson took up his position in the center, and on the 11th General Twiggs on the left, of the line of investment.

A pathetic incident of the landing was Captain Swift's effort to resume command of his company. After being ashore for three hours he fainted, was hurried to General Scott's steamer, thence to

New Orleans by the first sailing, at which place he died April 24, within a few hours after his arrival.

The investment was not completed until the 13th, on which day Twiggs extended his left to the sea.

The engineer company, with details from the line, was employed at first to open up a road through the chaparral to Patterson's right and thence along his line in the center. Upon completion of this work, the next undertaking was the destruction of the underground aqueduct which furnished the water supply for Vera Cruz.

From the completion of this task until the work upon the batteries commenced the company was engaged in reconnoitering between the line of outposts and the enemy's works. Meantime, Lieut. John G. Foster had been assigned to duty with it.

The location of the batteries for the heavy metal attack was done by the engineers, both staff and line, under the direction of Colonel Totten. General Wilcox states: "The engineers located and constructed the batteries with such good judgement and care, that there were few casualties." General Scott remarked to Colonel Totten after the reconnaissances had been made: "Vera Cruz must be taken with a loss not to exceed one hundred men; for every one over that number I shall regard myself as his murderer." The actual loss was 67. Some of the headstrong officers who held earthworks in contempt wanted to undertake an assault, but they were overruled by the general in chief.

Lieut. G. W. Smith in his official report states: "The work of throwing up batteries, digging trenches, and making covered communications with them, was commenced on the night of the 18th by large working parties detailed from the line. At that time the officers of the engineer company, including myself, were placed on general engineer service—supervising the construction of siege works. All the engineer officers then with the army, except the chief, were in regular turn detailed for that duty; each having some of the men of the engineer company to assist him."

Lee, G. W. Smith, and Tower, for instance, furnished each other relief in the construction of the naval battery. Much difficulty was encountered in all this work, owing to the fact that the northers often filled up the trenches with sand about as fast as it could be shoveled out.

The Mexican field defenses were constructed under the supervision of Lieut. Col. D. Manuel Robles, a very skilful engineer.

The American bombardment, however, was terrific and the city surrendered on March 27. Colonel Totten was appointed by General Scott one of the commissioners to treat for the capitulation. When that service was completed he returned to the United States.

After the surrender, the engineers were employed in making surveys of the city and its fortifications.

The advance into the interior was begun on the 8th of April with Twigg's division in the lead. Patterson's division followed on the 9th, and Worth's on the 13th. The engineer company was overlooked, but having gotten together an assortment of broken down animals it got under way on the latter date.

Some of the difficulties of the march may be gained from the following extract from Smith's report. "At Vergara half the load of each wagon was thrown out before we entered upon steep ridges and deep sand immediately after leaving the beach. All the men were engaged in helping along the half loaded wagons. That night we slept in the sand ridges.

"On the 14th, we reached Santa Fé, 8 miles from Vera Cruz, threw out the half loads, and returned to Vergara. Before we again reached the beach, the men had actually to roll the empty wagons up every hill, the mules not being able to drag them. By 10 p. m. we were again at Santa Fé, having killed three mules, and the men being worked nearly to death. Fortunately for us, several good mules that had escaped from preceding army trains came out of the chaparral to our feed troughs, were caught, and 'pressed' into engineer service." There are some compensations for bringing up the rear.

Meanwhile, Santa Anna was collecting his forces and constructing field fortifications at Cerro Gordo to dispute the American advance. Colonel Robles, his chief engineer, had reported against this position, pointing out that it could be turned in exactly the manner that Scott afterwards employed. He had recommended a stand at Corral Falso, some 7 miles in rear, but was overruled by his general.

General Twigg's division reached Plan del Rio at the foot of the pass on April 11. On the 12th daring reconnaissances were made by Capt. Joseph E. Johnston on the Mexican right and Lieut. P. G. T. Beauregard on the left. Both of these officers located the Mexican lines. Johnston received two severe wounds. Beauregard climbed Atalaya Hill and could see the Mexicans upon Telegrafo, the key to the position.

The reconnaissance begun by Johnston on the Mexican right was continued by Tower while Captain Lee made a number of close reconnaissances on the left, assisted by Beauregard, who managed to ascend almost to the top of Telegrafo. The interval from the 12th to the 17th was utilized by the engineers in this class of work. On the 17th Twiggs seized Atalaya and his command was known thereafter as the "Cerro Gordo Division."

General Wilcox remarks: "The information derived from these protracted reconnaissances determined the order of attack, which was to make a demonstration with two brigades, Pillow's and Shield's, on the Mexican right, and to throw the mass of the army around their left, thus cut their communications with the interior and capture or destroy their army."

The engineer company did not reach Plan del Rio until about 11.00 a. m. on the 17th. Lieutenant Smith immediately received orders to send a detail of ten men with an officer to report to General Pillow; McClellan was the officer selected. Smith, with the remainder of the company reported to General Twiggs. That night Smith and Foster constructed a howitzer battery upon Atalaya at a position selected by Lee. An 8-inch howitzer was emplaced the same night by Tower in the right rear of the Mexican line.

Of the battery located by Lee, General Wilcox remarks: "The artillery on Atalaya proved very effective, solid shot, shells, and rockets falling among the enemy on the top and along the slopes of Telegrafo, down into the valleys and deep ravines adjacent, in the camp, about Santa Anna's headquarters, and often far out among his reserves."

Riley's brigade, in its turning movement to the Mexican left on the 18th, was guided by Lee, assisted by Foster and ten men of the engineer company. General Shields having been ordered to support Riley, his column was taken under escort by Lee and led toward the Jalapa road.

In the attack on Telegrafo we are told: "Lieut. G. W. Smith, of the engineers, approached the hill top, reconnoitered the hostile line within pistol shot, and reported to Colonel Harney that there was no obstacle that could not be overcome without a halt." Telegrafo soon fell into Harney's possession. Smith, with his engineer soldiers, took part in the charge and killed two men with his own hands.

The Mexican account of the battle states that "The last effort was made then by Robles and the brave artillery officers."

Lieutenant Derby, of the Corps of Engineers, was wounded on Telegrafo.

General Wilcox in his summary of the battle of the 18th remarks: "Whilst awaiting the arrival and concentration of his troops, active reconnaissances were made daily and pushed forward with courage, led by intelligent engineers who appreciated fully the difficulties of the situation. He had ordered an examination of the whole Mexican front, even around his left, if practicable to the Jalapa road, and upon reports made by Captain R. E. Lee of explorations to that road, he based his orders of battle. The collision on the following morning (18th) demonstated his sound judgement, and the skill and courage of the subordinates, to whom he confided the execution of the order."

The American army resumed its march on the City of Mexico on the morning of April 19th with Twigg's division, preceded by the cavalry, still in advance. General Scott arrived at Jalapa on the 27th where he was delayed for several days in discharging the twelve months' volunteers. The criminal error of enlisting the first volunteers for this short period almost wrecked the commanding general's plans, just after his brilliant victory at the Pass of Cerro Gordo had opened the way to the capital itself.

The first detachments of the army left Jalapa on May 7th and Worth's division entered Puebla on the 15th. Here Scott was forced to remain until he could receive reinforcements.

While at Puebla the engineer officers were busily engaged exhausting all available sources of information of the Valley of Mexico. Both Major Turnbull, of the Topographical Engineers, and Captain Lee, of the Corps of Engineers, prepared maps of the same. These maps were compared and conferences held from time to time to determine in advance, if possible, the best front of approach to the city. Major Turnbull sent a man in his employ over the route to the south of Lake Chalco and it was reported practicable. Captain Lee is said to have fallen in with an Italian surveyor who assured him that the Chalco route was "available for the general's purpose."

The advance was resumed on August 7, 1847. The cavalry was in front of Twigg's division, which was preceded by the engineer company. From being in the rear with broken down

mules the latter was now in the lead with the finest transportation in the army.

The historian tells us: "General Scott, having with him the engineer officers, accompanied the leading division of the army on the descent into the valley and halted it at Ayotla, where he established his headquarters."

This place was well located for a thorough examination of the various possible routes into the city. The main highway from Puebla, over which the army had marched, led on westward to the city but it passed through the defile between El Peñon and Lake Texcoco. Shortly in advance of Ayotla, a road branched off to the left and at Mexicalcingo crossed the canal joining Lakes Xochimilco and Texcoco and led on to an intersection with the Acapulco highway running southward from the city. Between Ayotla and Peñon another road branched off to the right, and skirting Texcoco, joined with the highways approaching the city from the north. Immediately in rear of Ayotla was the rough road leading from the main highway around the southern shores of Lakes Chalco and Xochimilco to an intersection with the Acapulco highway at San Augustin.

On the morning of August 12, Captains Lee and Mason and Lieut. I. I. Stevens moved out with a heavy escort to reconnoiter El Peñon. Maj. John L. Smith and Lieutenants Beauregard and Tower examined the road leading around Lake Texcoco. On the afternoon of this date, Lee and Beauregard examined the north shore of Lake Chalco as far as San Francisco. They here confirmed the information obtained at Puebla as to the practicability of the road around the southern shores of the lake.

On the 13th, Mason, Beauregard and McClellan proceeded to reconnoiter Mexicalcingo. Stevens returned to El Peñon. Lee and Tower again examined the northern shore of Lake Chalco and then rode forward to join the party at Mexicalcingo.

On August 12, General Scott sent verbal instructions to General Worth to have a reconnaissance made around the southern shores of Lakes Chalco and Xochimilco. Written orders to the same effect were sent on the 13th. The reconnaissance was made on the 14th and the route reported practicable.

The engineers reported that "El Peñon could be taken, but with a great and disproportionate loss." They also reported that the canal could be forced at Mexicalcingo, but that the narrow

causeway beyond flanked on each side by swampy ground would prove a great handicap to this route.

In justification of his final action General Scott in an official report said: "Those difficulties closely viewed threw me back upon the project long entertained of turning the strong eastern defenses of the city by passing around south of Lakes Chalco and Xochimilco, at the foot of the hills and mountains, so as to reach this point, and hence to manœuver on hard ground, though much broken, to the south and southwest of the capital, which has been more or less under our view since the 10th instant. We are told that 'Santa Anna and his generals were greatly disappointed in not being attacked at the Peñon, where they were well prepared, and where the skill of the engineer, Robles, exerted in vain, was expected to accomplish great results.' "

This reconnaissance before the city to determine the front of attack is dwelt upon rather at length because of the historic controversy to which it in part gave rise. In the "Tampico letter" Lieutenant Colonel Duncan, of the artillery, had claimed all the credit for the move to the south of the city for General Worth and himself. At about the same time appeared the letter published in the *New Orleans Delta* giving General Pillow all the credit for the victory at Contreras. When these papers reached the City of Mexico, about two months after the surrender, General Scott published a scathing order charging these (at that time unknown) authors of "pruriency of fame not earned." This order at once brought to light the authors and led to charges being preferred by General Scott against Generals Worth and Pillow, and Lieutenant Colonel Duncan. General Worth was not in any way connected with the "Tampico letter." His trouble with Scott had arisen at Puebla, where Scott had reprimanded him, and where a court of inquiry, far from exonerating Worth, had "rubbed it in" still deeper. He felt Scott's order directed at him and became involved.

This controversy became far reaching in its effects. General Scott himself was relieved of his command and ordered before a court of inquiry. History has done the old general full justice, and his fame grows brighter as the days pass.

The movement to the south of the city began on August 15 with Worth's division, preceded by the cavalry and engineer company in the lead. The road, which in places was at the water's edge and at the foot of high hills, afforded the engineers sufficient

trouble. A working party of 500 men was detailed from the line and was supervised by the engineer officers and men under the protection of the infantry.

The American advance column reached San Augustin on the afternoon of the 17th. On this date General Valencia had arrived at San Angel and pushed forward the engineers, Cadena and Segura, with General Mendoza and Don José Mario, to make a thorough reconnaissance of the neighborhood of Padierna. Late in the afternoon Captain Blake of the Second Dragoons, accompanied by Captain Mason, of the engineers, proceeded to Peña Pobre toward Padierna and the party was reported to Valencia.

On the 18th began the thorough reconnaissances to determine the best route into the city. General Scott now had plenty of elbow room in which to maneuver. Mason, Stevens, Tower, G. W. Smith, McClellan and Foster, accompanied by a small escort under Capt. Seth B. Thornton, of the Second Dragoons, moved straight ahead toward San Antonio on the Acapulco highway. The party had not gone far before it was fired upon by a Mexican battery and Captain Thornton killed. Lee and Beauregard, with an escort of two companies of infantry and Capt. Phil Kearny's company of dragoons, made an examination of the country toward Padierna.

The Mexicans had strong works at San Antonio, and as the road thither was flanked on the one hand by the Pedregal and on the other by swamps, the reports of Mason and Stevens were not very favorable to this route.

The reconnaissance of Lee and Beauregard was more favorable. They found a road practicable for artillery for a mile over the Pedregal, and reported that it could be extended entirely across with a suitable working party, and a way opened for a further move to the left. These officers also reported that the Mexicans were occupying Padierna in force with artillery emplaced.

General Mendoza, the ranking member of the Mexican reconnoitering party heretofore referred to, had reported that Padierna was absolutely indefensible and recommended a withdrawal to General Valencia. In spite of this obviously sound advice, and a further direct order from Santa Anna, he obstinately held on.

With the information at hand General Scott decided to mask San Antonio with Worth's division, and move in force toward

Padierna, leaving Quitman to guard the supplies at San Augustin. Pillow's division was to furnish the working party of 500 men for the road work while Twigg's division was to cover the operations; Pillow was to support Twiggs in case he became heavily involved.

At 3.00 a. m. on the 19th Smith, attached to Worth's division with the engineer company, received an order to return to San Augustin with the engineers and train. While wending their way back through Worth's weary command, the engineer officers heard one line soldier remark that Worth was not going to fight because the engineers were being detached. McClellan remarked to Smith, "I consider it the handsomest compliment that could be paid to the engineer company. The private soldiers of this army understand that we are sent where the hardest work and hardest fighting are to be done—and always at the head of the leading division."

The company was reported to Capt. Robert E. Lee, and two parties were formed under Smith and McClellan to supervise work on the Pedregal road under the direction of Lee himself.

About 3.00 p. m. the working parties reached a point where they came under fire of some 22 pieces of artillery. Capt. John McClellan and Lieut. George B. McClellan, having ridden forward, were fired upon by the advance posts of the enemy. A section of Magruder's battery was conducted into position by McClellan and the mountain howitzer battery by Foster. General Smith successfully attacked the advance works of the enemy at Padierna and the engineer company took part in the action.

Riley, in his turning movement to San Geronimo, was accompanied by Tower; while Colonel Morgan's regiment, followed by Shield's brigade, was conducted toward the same place by Beauregard. General Smith later moved to the same point, conducted by Smith with the engineer company. From the vicinity of San Geronimo important information of the enemy and routes of approach was obtained by the three engineer officers. It was on this night that Robert E. Lee came to General Smith's headquarters to find out for General Scott the exact state of affairs in his detached command, and made his famous ride back to headquarters over the Pedregal.

The columns of attack were conducted on the morning of the 20th by Smith, Tower and Beauregard, over the routes previously selected by them. The engineer company took part in the successful surprise attack upon Valencia's right rear.

In the pursuit of the enemy Smith's brigade, preceded by the engineer company, led the way. When Coyoacan was reached General Scott sent forward Lee with a strong escort to reconnoiter. Lieuts. G. W. Smith, G. B. McClellan and I. I. Stevens were sent forward toward Churubusco with the engineer company on a similar mission.

When Worth attacked San Antonio in front and sent Clarke's brigade over the Pedregal to turn the right of the position the latter column was conducted by Mason and Hardcastle; Lee, in his reconnaissance, rode toward San Antonio until he met Clarke. He then returned to General Scott and informed him that Shield's brigade, which had gotten in rear of the Mexican position at Churubusco, was encountering great opposition. Reinforcements were then sent to Shields under Lee's guidance.

Smith, Stevens and McClellan, in their reconnaissance, were soon upon the strong position at the Convent and sent back valuable information. Toward evening the Mexicans were driven into the City of Mexico, and thus ended in victory on the 20th of August, 1847, the battles of Contreras and Churubusco.

General Scott now moved once more to his left, established his headquarters at Tacubaya, and after having gotten his divisions located in favorable positions, entered into a general truce with Santa Anna. Nothing came of the peace negotiations between Mr. Trist and the Mexican commissioners.

The engineers now began a systematic reconnaissance to find a way into the city.

On September 7 Lee examined the San Augustin road. In the morning of this date Mason made a close reconnaissance of Molino del Rey and found the position strongly held by the Mexican infantry and artillery. In the afternoon he continued the work, in company with Duncan, of the artillery, and Hardcastle, to select positions for the artillery. Beauregard was engaged in making an examination of the Niño Perdido and San Antonio causeways.

General Scott decided to attack the position at Molino del Rey. The feature of the battle was Major Wright's storming party of 500 picked men. The fight for the possession of the buildings on the 8th was a very bloody affair. Of the fourteen officers in Wright's command, twelve were either killed or wounded. Two of the severely wounded were Mason and Foster, of the engineers. After two hours of hard fighting, in which

the American casualties were 787 out of 3,100—about 25 per cent—the Americans stood in possession of the field.

After General Scott had received such unexpectedly strong opposition at Molino del Rey he became exceedingly careful in his selection for the next point of attack, which he meant to press within the gates of the city with his entire force.

On August 8 Lee, Tower and Beauregard reconnoitered as far as Piedad and recommended that it be occupied. These officers then made a close examination of the Piedad and Niño Perdido roads.

On August 9 the commanding general himself, accompanied by Lee, reconnoitered the San Antonio causeway and the works across it at the entrance to the city.

Repeated examinations on successive days were made of the causeways leading into the city by different engineer officers. Particular attention was paid to the San Antonio Garita which seemed to offer a favorable point of attack.

On August 11 General Scott held a council of war to determine upon such a point. At this council were present Generals Scott, Pillow, Quitman, Twiggs, Pierce, Cadwalader and Colonel Riley. The engineer officers present were Maj. John Lind Smith, Captain Lee and Lieutenants Beauregard, Stevens and Tower.

General Scott informed the council that he intended making an attack soon and that the point selected rested between the San Antonio Garita and Chapultepec, and thence on to the Belen and San Cosme Garitas.

Pillow favored the San Antonio route. "Quitman, not having been near the city, not being well informed, and claiming slight knowledge of the art of war, would express no opinion until he had heard the report and opinions of the engineers."

Major Smith, Captain Lee and Lieutenants Tower and Stevens reported their observations and advocated the San Antonio gate. Agreeing with them were Quitman, Shields, Pierce and Cadwalader. Generals Scott and Twiggs were in favor of the Chapultepec route. Colonel Riley finally agreed with them.

"Lieutenant Beauregard was the only engineer who favored the Chapultepec route. He had examined the San Antonio gate frequently, commencing on the night of the 7th, had seen the works strengthened by night and by day, knew that the attacking columns would be exposed to full view, and that the ground over

which they would have to advance was intersected by many irrigating ditches, some of them deep, and much of the ground under water. He gave reasons why Chapultepec should be assaulted, and General Pierce, after hearing them, changed his opinion. General Scott, having heard the opinions of all, said: 'We will attack Chapultepec, then the western gate;' and the meeting was over."

Captain Huger, of the ordnance, selected four sites for batteries to attack the strong castle of Chapultepec. Battery No. 1 was constructed under the supervision of Lieut. G. W. Smith, and No. 2 under G. B. McClellan. Battery No. 3 was constructed under Lee's direction.

The bombardment was begun on the 12th and continued on the 13th of September.

The assaulting column from General Quitman's command, under Capt. Silas Casey, was guided forward by Lieutenant Tower in person, who was wounded.

In the pursuit of the enemy from Chapultepec, Lieutenant Stevens was severely wounded while reconnoitering in front of the Voltigeurs on the San Cosme causeway. Lieut. G. W. Smith was the next engineer officer in rank with Worth's column, now approaching the city from the west. Smith was ordered forward to carefully examine into the state of affairs and to report upon the best method of conducting the attack, especially in regard to the use of siege guns. Smith reported that the gate could be taken by infantry alone and proposed a method. He was ordered by Worth to carry out the plan suggested with the engineer company, and line officers were directed to give him all the needed assistance. G. B. McClellan was with the company at this time.

"Lieutenant Smith, ascending to the roof of a one-story house, found steps leading to the roof of a building one story higher near the enemy's battery, and from which the interior of his works was exposed to view. About twenty infantrymen and some from the engineers, mounting the steps one at a time, ascended the roof of the two-story house; the position of the enemy below was pointed out to them, and concealing themselves behind roof walls until their numbers had been increased sufficiently to make their fire effective, the enemy meanwhile unconscious of what was going on above and so near, they were then ordered to fire, and the Mexicans being 40 yards, the charge was most deadly.

. . . Passing the battery at the Garita a lodgment was effected in houses on the left and, placing the main portion of the force under shelter, Lieutenant Smith moved on with the engineer company and a detachment of the Fourth Infantry. . . . Lieutenant Smith, whilst utilizing the technical knowledge of the engineer, had displayed both courage and good judgment in directing the above movement."

In the advance through the city on September 14 the Mexican "snipers" were very active in picking off American officers and soldiers. The engineer company was at the head of Worth's division and assisted in clearing the streets and houses. Sergeant Hastings, of the company, having been shot, McClellan took his musket and killed his assailant.

Quitman's division was the first to enter the Grand Plaza. As soon as it arrived there, Beauregard was sent to General Scott to acquaint him with the fact. The General had held Worth in leash and seemed very anxious that Quitman should have the honor of first planting the Stars and Stripes in the Plaza.

A few interesting facts generally well known are connected with the subsequent careers of some of the engineer officers who participated in the Mexican War.

Beauregard contested with his classmate McDowell (of the infantry and staff) the First Bull Run

Joseph E. Johnston and McClellan struggled at Seven Pines until the former was wounded, when Gustavus W. Smith for a few hours sought to overwhelm his old subordinate of the engineer company.

Lee then contested with McClellan, Pope and Meade the Seven Days' Battles, Second Bull Run, Antietam and Gettysburg.

Two of the officers of the Corps of Engineers prominent in Mexico were killed on the field of battle as general officers during the Civil War.

Gen. Isaac I. Stevens was killed at Chantilly on September 1, 1862, attempting to check Jackson's turning movement along the Little River Turnpike immediately after the Second Bull Run. McClellan's campaign on the Peninsula had ended in failure; Pope's headquarters were being transferred from "the saddle" to a bombproof in the defense of Washington. At this crisis the President was casting about to find a successor for both of these generals, and Stevens' name is said to have been under consideration.

Only sixteen days later at Antietam fell General Mansfield at the head of the Twelfth Corps. Although he had many premonitions of death during the Mexican War he escaped, but he was severely wounded at the Tannery at Monterey, where he led the assault.

In 1851 Stevens published a short work on the Mexican War in which we find a few passages that have a familiar ring. He tells us: "A grave error had been committed in the army reduction of 1842, making the companies mere skeletons, and reducing the whole number of bayonets and sabers to less than eight thousand. This small force, moreover, had been disseminated through the country in small garrisons, a condition utterly unpropitious to high discipline and efficiency. There had been few general camps of discipline, where all arms had met together to learn the military art. In consequence of this system of petty garrisons, and this neglect of camps of discipline, regiments were deficient in the regimental drill, and practical instruction in evolutions of the line was almost unknown. . . . The army had been regarded more as a police force to maintain forts and guard our Indian frontier than as an embodiment of the military skill and science of the country."

One is almost forced to close this article with the old quotation: "There were giants in those days."

The Flow of Sediment in the Mississippi River and Its Influence on the Slope and Discharge.*

WITH ESPECIAL REFERENCE TO THE EFFECT OF SPILLWAYS IN THE VICINITY OF NEW ORLEANS, LA.

BY

Col. C. McD. Townsend,
Corps of Engineers, U. S. Army; President Mississippi River Commission

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The investigation of the influence of spillways in the vicinity of New Orleans, on flood heights, which has been directed by the Commission, renders it necessary to determine: First, the changes in river section which will result therefrom and their effect on navigable depths; second, the resultant reduction in flood heights; and third, the amount of sediment that will escape through the spillways, and the effect of the deposition of this sediment on the carrying capacity of the spillway. In other words, it necessitates an investigation of the flow of sediment in the Mississippi River, the formation of bars therefrom, and the influence of these bars on the river's slope and discharge.

Humphreys and Abbot made a valuable series of investigations on the flow of sediment in the Mississippi River at Carrollton in 1851-1852, and at Columbus in 1859. A Board of Engineers began a similar series of observations in 1879, which, upon the organization of the Mississippi River Commission was extended by that body so as to cover the flow of sediment in the river from Prescott, Wis., to the mouth. The Geological Survey, in 1906-1907, also investigated the flow of sediment in the Missouri and Mississippi, among other rivers. In addition to the above, there have been numerous investigations of the flow of sediment in the rivers of Europe. The early observations are summarized in a paper by E. H. Hooker on the suspension of solids in flowing water, published in Vol. XXXVI, Transactions of the American Society of Civil Engineers. Since that paper was prepared there have been published in the proceedings of the various International Navigation Congresses the results ob-

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*Paper presented at the 128th Session of the Mississippi River Commission.

tained by other investigations, of which the most noted are those of M. Fargue on the Loire, M. Girardin on the Rhône, and M. Leliavski on the Dnieper.

It is believed that an analysis of these observations will be of value, not only in solving the problems arising from the construction of a spillway, but others which are presented to the Commission.

The writer has been a member of several boards and committees of engineers who have had to deal with kindred topics, and as he is being extensively quoted and misquoted at the present time, he considers it advisable to place on record his own views on these subjects, unfettered by the necessity of bringing them into harmony with those of others.

Material is carried down a river in solution, in suspension, and by being rolled along the river bed. That in solution is carried to the Gulf without deposition, and does not affect the problem under consideration. It may, however, be noted, that not infrequently a stream carries more material in solution than in suspension, conspicuous examples being the outlets to the Great Lakes. The observations of the Geological Survey demonstrate that at Minneapolis the material in solution is greater than that in suspension, the mean of the observations showing 200 parts per million in solution and 7.9 parts per million in suspension.

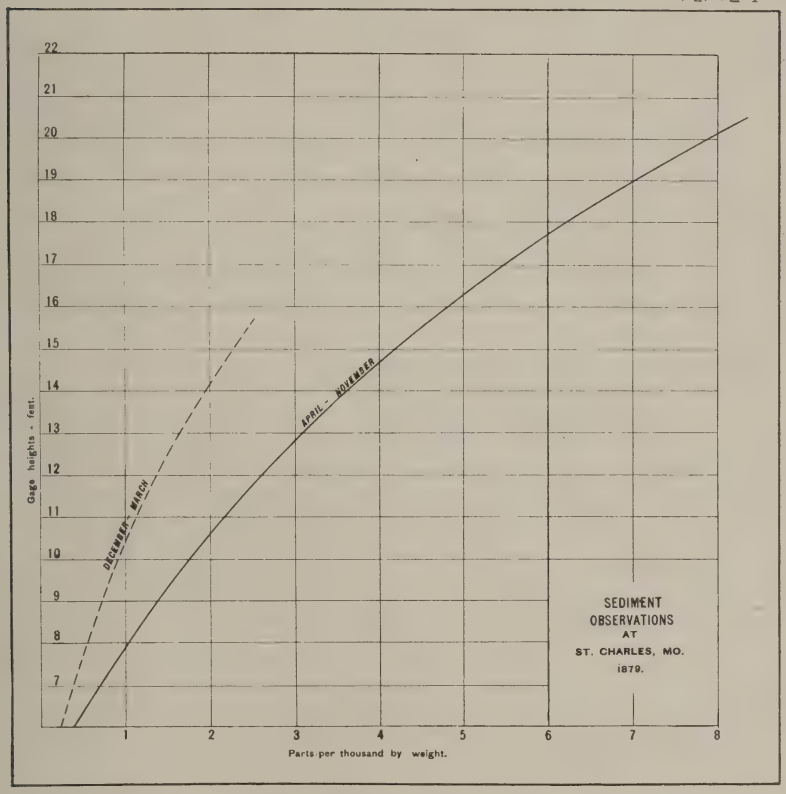
The observations on material carried in suspension by the Missouri River, made at St. Charles, Mo., in 1879, indicate that the amount of sediment per unit of volume of water (called hereafter the degree of saturation) is a function of gage heights. The curves shown in Plate I give the mean curves of saturation for gage height—the full line from April to November, and the broken line for winter months. The variations from these curves do not exceed those which are observed when a mean discharge curve is plotted.

The observations of the Geological Survey in 1906-1907, at Ruegg, which is about 19 miles below St. Charles, conform to these curves with the exception of the set of observations from June 16 to June 25, which give a degree of saturation markedly less than shown by the winter curve.

The observations of the Geological Survey on the Missouri River near Florence, Nebr., and Kansas City, Kans., indicate that similar curves of saturation could be constructed for those localities, but that the degree of saturation would be greater for similar gage heights.

In the Mississippi River, however, no relation between gage heights and amount of sediment in suspension is apparent. The observations at Prescott, Minn., show that considerable material is carried by suspension into the Mississippi from the Minnesota River, but it is deposited in Lake Pepin. At Winona, Minn., and Clayton, Iowa, the material carried in suspension is insignificant.

PLATE I



At Hannibal, Mo., and Grafton, Ill., the degree of saturation at the same stage may at one time be ten times greater than at another, dependent on the source of the flood, and the maximum degree of saturation is about one-sixth that of the Missouri at St. Charles, Mo.

The sediment from the Missouri River so largely exceeds that from any other tributary of the Mississippi, that below its mouth the degree of saturation appears to be principally dependent on the proportion of water from that river that is flowing down the

main stream, and the total amount of sediment moving per day closely conforms to the amount it discharges, as is shown on Plate II, on which are plotted the amounts of sediment observed per day at St. Charles, Mo., St. Louis, Mo., and Columbus, Ky., in 1879, after making due allowance for the time required for the flow of water between the localities. A similar tendency is observed at Helena, Ark., and is also apparent in the observations at Fulton and Carrollton in 1880, and in those by the Geological Survey in 1907 on the Missouri River and at Memphis, Tenn.

The sediment observations at St. Louis in 1879, which are summarized in a paper by Robt. E. McMath, published in Vol I, *Journal of the Association of Engineering Societies*, are particularly valuable, as they were taken 11½ miles below the junction of the Upper Mississippi and Missouri Rivers, and at the same time as those at St. Charles on the Missouri, and at Columbus, 21 miles below the mouth of the Ohio, with which they can be compared.

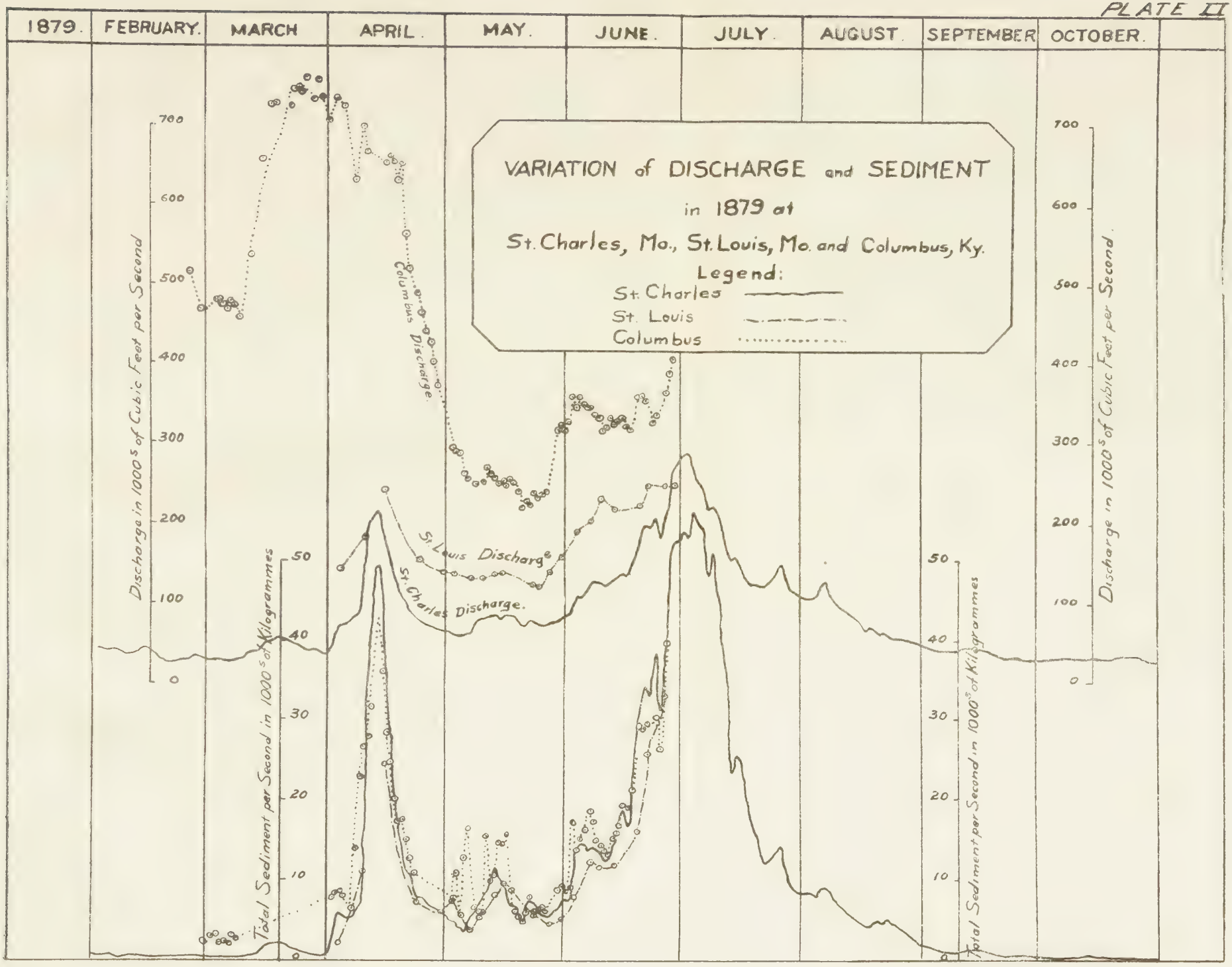
They show that at low stages the water of the Missouri flows along the Missouri side of the river, and that of the Mississippi along the Illinois side, while at high stages there is a more intimate mixing of the waters, and at certain intermediate stages the slightly saturated water of the Mississippi appears to be flowing above the more saturated water of the Missouri. The maximum amount of sediment is, however, usually found near the bottom on the Missouri side of the river, and the minimum at the water surface on the Illinois side.

In addition to material moving without losing contact with the bottom, Mr. McMath calls attention to "material which, though heavy and in grains of notable size, is detached for a time by some energetic impulse and describes a longer or shorter free path, moving in or out with the surrounding water," and, "material which once mingled with the water remains in continuous suspension until it reaches the sea."

Lieutenant Leach, in his analysis of the observations at Carrollton, also states that sediment may be classified as permanently or intermittently in suspension. The amount of the former at any given time will depend almost wholly upon the proportion of the river's discharge which has been contributed by the Missouri River. The amount of the latter will depend upon the extent and violence of the local action in the vicinity.

1879.

PLATE II


1879.
FEBRUARY.
MARCH
APRIL.
MAY.
JUNE.
JULY.
AUGUST.
SEPTEMBER
OCTOBER.
VARIATION of DISCHARGE and SEDIMENT
in 1879 at
St. Charles, Mo., St. Louis, Mo. and Columbus, Ky.
Legend:
St. Charles
St. Louis
Columbus
Discharge in 1000s of Cubic Feet per Second
Total Sediment per Second in 1000s of Kilogrammes
Columbus Discharge.
St. Louis Discharge.
St. Charles Discharge.
700
600
500
400
300
200
100
0
50
40
30
20
10

When the highly saturated Missouri River water mingles with that of other rivers, notwithstanding the enormous caving of banks which occurs on the Lower Mississippi, it never again attains the degree of saturation it had above St. Louis, and water from the Ohio River never attains one-third the degree of saturation that is found at high stages on the Missouri.

The observations at Carrollton indicate that during floods from the Ohio River, the relation of the amount of sediment carried by the surface waters to those flowing near the bottom, is as 1 to 1.83; during floods from the Missouri the amounts are more nearly equal.

When the velocity of water highly charged with sediment is suddenly checked, there is a large deposit. In the Report of the Chief of Engineers for 1880, page 1364, it is stated that at Horse-tail Bar, 7 miles below Eads Bridge, permeable dikes placed in the Mississippi River caused in a year a maximum deposit of 64 foot depth, and an average deposit of 10 feet. Material thus deposited is not as readily again carried in suspension as when washed into the river by rains, but when disturbed tends to move down the river by being rolled along the river bed.

From the above it is deduced that material carried in suspension down a river is primarily dependent on the character and condition of the soil in the river's watershed, and the intensity of the rainfall over it. Material thus brought into the river moves with a velocity equal to that of the water, and is transported to the sea unless the current is suddenly checked, when a portion is deposited. An increase in velocity will again set this material in motion, but most of it will then move down the river in sand waves.

Mr. Arthur Hider, in his investigations of the movement of sand waves in Lake Providence Reach, deduces the following: that material moves along the river bottom in sand waves, which are functions of the velocity of the current and the depth of water, being the greatest when for any cause the current is suddenly increased, at which time the most rapid erosion of the bottom takes place, and conversely, the movement of material along the bottom is least when the velocity of the current is suddenly decreased, as then the greatest deposit of sedimentary matter occurs. These waves have an irregular motion down stream, and the maximum size and rate of progress is attained when the river is at its highest stage and nearly stationary; their height, length and

rate of motion being dependent on their location with reference to the line of maximum velocity. The waves have the least dimensions and slowest rate of travel at low water. They move down stream by material being pushed or rolled up their flat anterior slope, and being dropped over their crest where it remains until the waves have progressed far enough down stream to expose it again to the action of the current, to be again rolled or pushed forward. The amount of material thus moving is greatest in high water, or when the velocity for any cause has been suddenly accelerated. Changes in the form of waves are gradual, the waves retaining their form and individuality so long as the velocity of the current remains nearly uniform. They are destroyed by erosion if there is a rapid increase in velocity, and are obliterated by deposition of sediment if the velocity is suddenly decreased. They again make their appearance when the velocity of the current approaches uniformity for any length of time. Sand waves were observed in 90 feet of water, having an average length of 750 feet, a maximum height of 22 feet and a maximum rate of travel of 40 feet per day.

At Plum Point Reach the average length of sand waves observed was 600 feet, extreme height 12 feet, maximum daily motion 35 feet.

At Helena, when the maximum depth of water was 30 feet, the maximum length of sand waves was about 400 feet, extreme height 8 feet, and rate of motion of crest about 20 feet per day. These waves did not extend entirely across the river, but disappeared about 600 feet from each shore.

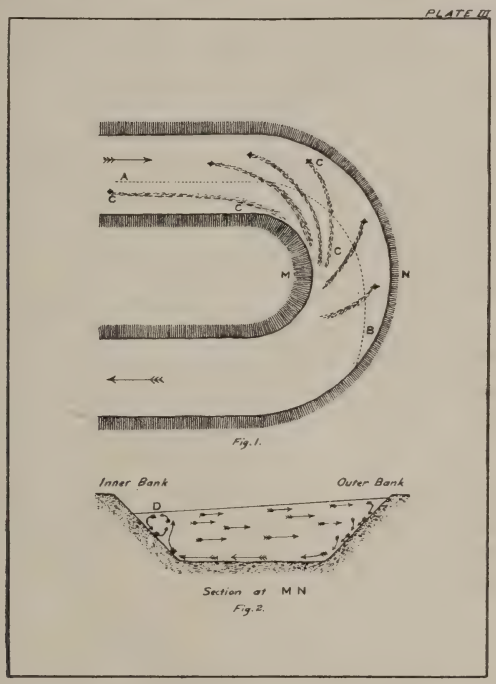
At Carrollton the maximum length of waves observed was 1,000 feet, maximum height 15 feet, maximum velocity 25 feet.

At Carrollton the amount of material being transported in sand waves during the year was estimated at less than 1 per cent of that carried in suspension; at Lake Providence Reach about 10 per cent.

In addition to this movement of sand waves downstream, there is a movement of material across a river bed, which has not received the attention from American investigators that its importance merits.

In a paper read before the Institution of Mechanical Engineers at Glasgow, in 1879, Professor James Thompson, F. R. S., invited attention to the fact that while in bends of rivers, the caving occurs along the concave bank and a corresponding fill is formed

on the opposite side, the surface currents at the time of greatest caving are greater along the convex than the concave banks, and that in straight reaches of the river the greatest erosion occurs where the velocity is the greatest. His explanation of this apparent discrepancy was that in bends there is constrained motion, which causes the water to pile up on the concave side of the bend, and produces an appreciable difference of elevation of the water surface on opposite sides. Under the action of this



centrifugal force, and difference of levels, the water is not only forced against the concave bank, but the layers in contact with it have a downward motion to the bottom of the river, thence they move across it, rising to the surface on the convex side.

To verify this theory he constructed on a small scale an artificial channel, including a bend, in imitation of a river. This apparatus was exhibited in an adjoining room, and the phenomena were shown by the use of seeds serving as detritus, and small objects floating on the surface; and also in a new way by little specks of aniline dye introduced so as to adhere to the bed of the channel at various places on the bottom and banks under

water. From each speck of the adhering dye the flowing water carried forward coloring matter, which showed that the course of the thin film of the stream, close to the bed, was along such lines as CC in the plan, Plate III, Fig. 1, the stream there flowing strongly towards the convex bank; while a floating object placed on the water surface would move along a line such as AB, in Fig. 1, going towards the concave bank. The whole of the upper part of the river flowed outwards slowly, while the stream along the bottom flowed inwards very quickly. There was a little eddy at D, Fig. 2, whose existence was manifested very clearly by the aniline dye. The theoretical considerations connected with this eddy were very interesting; but for brevity he did not enter upon their discussion on that occasion.

In a paper presented to the Sixth International Navigation Congress, held at The Hague, in 1894, M. Leliavski describes a submarine vane invented by him to determine the direction of subsurface currents, and which he employed on the Dnieper to verify in a river the above theory. From the results of his observations he comes to the conclusion that in a bend, surface currents converge toward the concave bank, along which a stream of water flows to the bottom of the river, thence moves to the convex side in divergent currents, and then gradually rises to the surface.

These currents have sufficient force not only to cause caving on banks of clay, sand or gravel, but he found evidences that they had caused erosion in the lava rock which forms the bed of the river at the Dnieper rapids.

The greatest depth was found where the greatest number of surface filaments of water converge on the concave bank, and if the radius of curvature of the bend is uniform, some distance below its middle point. The eroded material was carried across the river by the divergent currents and deposited in a fan-shaped bar.

He considers the sand waves moving down a river in a bend the residuals of this cross-stream motion, but observations on the Upper Mississippi River have demonstrated that sand waves obstructive to navigation are found in straight reaches where a centrifugal force can not be developed, and also move through bends of gentle curvature.

These experiments and observations afford a satisfactory explanation not only of the excessive caving that occurs in bends, but of the fact that with this caving the area of cross section of

a river does not materially increase. They account for the immense mass of material which falls into the river from caving banks, and is not apparent in material moving in suspension or by sand waves.

All non-tidal rivers flowing in a soil susceptible of erosion by their currents, follow sinuous lines and have relatively deep pools usually in the concave sides of their bends, separated by higher ridges or natural dams called crossings.

A cross section of the pool is usually triangular, and that of the crossings trapezoidal. The crossing extends diagonally across the river from the lower end of the bar on the convex side of one bend to the beginning of the similar bar in the bend below, and has a length on its crest greater than the width of the pool.

These forms necessarily result from the motion of water described in the paragraphs above. They exist on the largest rivers but are modified by the character of the soil and by the volume and velocity of the river's discharge.

In light, sandy soil the rate of caving in bends largely exceeds that which exists when the soil is more tenacious. The concave bank assumes a more gentle slope and the channel or thalweg depth at that locality is less. If, however, the bank be revetted so as to prevent further caving, the thalweg depth increases. The form of the bend has, however, a greater influence on thalweg depths, and within certain limitations, the deeper the bend or sharper the curvature, the greater becomes the thalweg depth.

From observations on the Loire, M. Fargue, in a paper presented to the Fifth International Navigation Congress, sums up the matter as follows:

The channel or thalweg follows the concave bank, and sand bars are deposited on the convex. The channel is so much the deeper and the sand bank so much the more salient, as the curvature is the more accentuated. The maximum and minimum depth correspond, respectively, with the maximum or minimum of curvature.

This agreement does not, however, take place in the same transverse profile. The deepest water is below the point of greatest concavity, and the greatest sand bars are below the point of greatest convexity. The shoal water on the crossing is below the point where the concavity of the bank changes to convexity.

The channel presents regularity in its longitudinal profile only where the axis of the bed varies in a manner gradual and con-

tinuous, and every abrupt change of curvature is accompanied with an abrupt change of depth.

These relations exist only in the portions of the river where the length of the curves, that is to say, the distance between two consecutive points of inflection, are neither too great nor too small. Whenever these relations do not exist the channel is formed of isolated pits separated from each other by ridges which promptly reform when removed by dredges.

The radius of curvature of the bends is also affected by the volume of the discharge. Thus at St. Paul, where the discharge at low water is about 2,000 second feet, and at high water about 80,000, the radius of curvature in the bends is from 1,600 to 4,500 feet, while in the bends above Greenville, Miss., it is from 8,000 to 15,000 feet.

The Atchafalaya River affords an interesting example of the influence of discharge on the form of the river bed, as described by Mr. J. A. Ockerson in a paper on the Atchafalaya River in Vol. LVIII, Transactions of the American Society of Civil Engineers.

Originally the Atchafalaya was obstructed by an accumulation of snags and drift called a raft, which limited the amount of water that could flow through it, both at high and low stages. The removal of the raft and the construction of levees along its banks has largely increased its discharge, both at high and low water, and as a result the river has attempted to enlarge its section, but instead of retaining its old sinuosities and forms it has created new ones, cutting a channel through bars on convex points, and thus attempting to adjust its curvature to its discharge. It is, however, very pertinent to the discussion to note that while the river has sufficient force to excavate a new channel adapted to its discharge, it does not carry sufficient sediment to fill up the old forms which it has abandoned.

In the Illinois River, where the low water discharge has been increased from about 1,000 second feet to over 5,000 by the flow from Lake Michigan through the Chicago Sanitary Drainage Canal, a corresponding change in the radius of curvature of its bends is also taking place.

When the river passes from low water to a flood stage there is an effort to similarly adjust the radius of curvature to the volume of discharge. If the bend followed by the current at low water is too deep, as in Plate IV, Fig. 3, the subsurface currents in the bend deposit material on the outer edge of the bar, while

PLATE IV

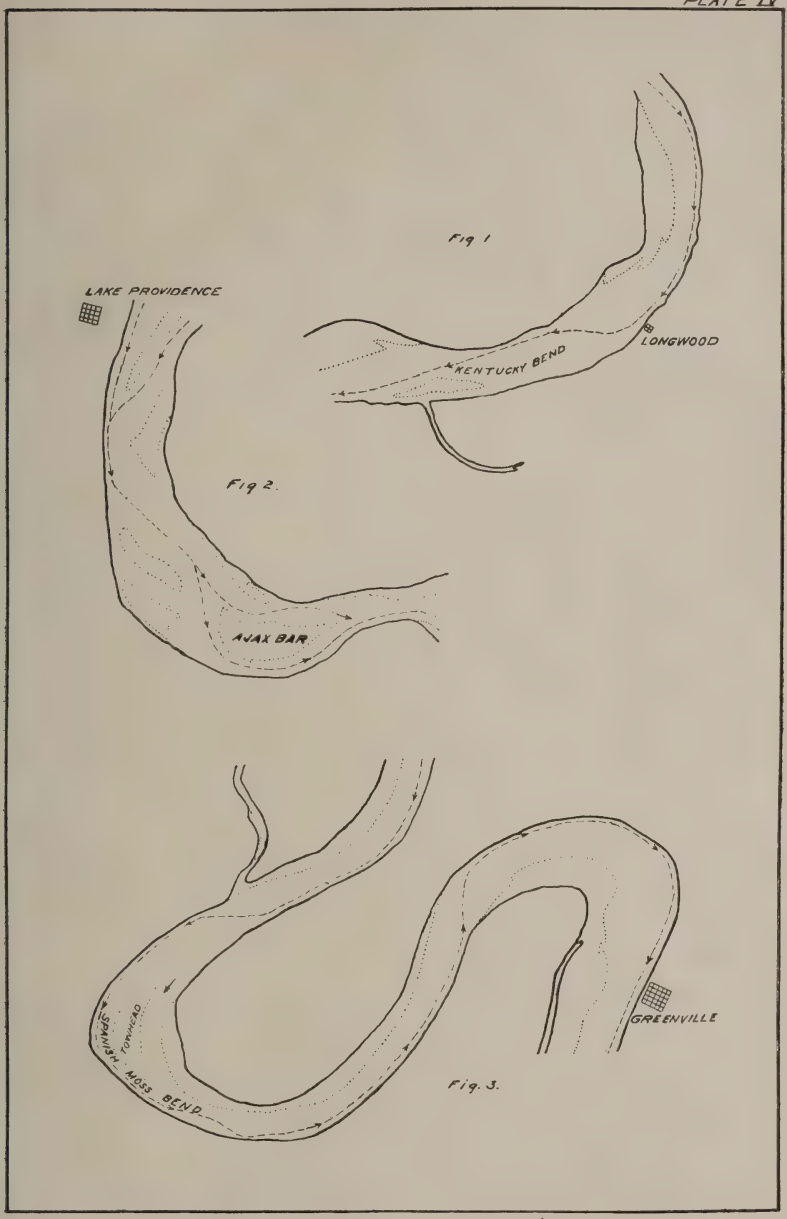


Fig 1
LONGWOOD
KENTUCKY BEND
LAKE PROVIDENCE
Fig 2.
AJAX BAR
GREENVILLE
SPANISH MOSS BEND
TOWHEAD
Fig. 3.

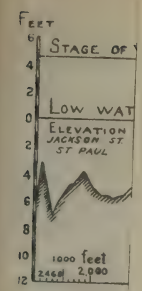
the surface currents, as the river rises, flow across the point beyond their sphere of influence. As a result, these surface currents soon excavate a channel, leaving an island on the extremity of the point, called on the Lower Mississippi a towhead, and the main currents of the river during floods flow through this channel of gentler curvature.

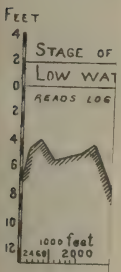
Also, if during floods the currents have created a channel in a bend of too gentle a curvature (see Fig. 1), it may happen that as the river falls the centrifugal force is not sufficiently developed to excavate a channel along the bend, deposit takes place, and the low-water thalweg, after following the bend for a certain distance, moves over to the opposite side. Fig. 2 is an illustration of another cause of the thalweg not following the bend; that is, the influence on the currents of islands and a divided channel above it.

In the Upper Mississippi, where the banks are more stable, caving does not occur at low stages, and the volume at mid-stage has a greater controlling effect on the radius of curvature and the bends have greater stability than on the lower river where the soil is so friable that caving occurs at all stages, and the locus of maximum caving changes its position with the stage, and may even be transferred from one bank to the opposite.

A change in the mean slope of the river appears to have a greater effect on the height of the crossing than on the shape or depth of the pool. In Plates V and VI are shown thalweg depths just below St. Paul, Minn., below the Chippewa River, below Quincy, Ill., and through Lake Providence Reach, where the mean slopes are about four-tenths of a foot to the mile. In Plate VII are shown thalweg profiles of stretches of river below those in Plates V and VI where the mean slope has been reduced to about two-tenths of a foot per mile.

At low stages of the river the area of cross section of the pool largely exceeds that of the crossing, and as the same quantity of water must pass through each section it flows through the pools with gentler slope and much less velocity than over the crossings. There is then a tendency for the crossing to scour and the material thus moved to be deposited in the pool below. As the river rises the area of cross section of the river at the crossing increases more rapidly than at the pool. Erosion then begins in the pool, at first supplying sufficient material to the crossing to compensate for what is removed therefrom, and checking the process of channel deepening. As the rise continues the material


FEET
STAGE OF
LOW WAT
ELEVATION
JACKSON ST
ST PAUL
1000 feet
2468 2,000


FEET
STAGE OF
LOW WAT
READS LOG
1000 feet
2468 2000

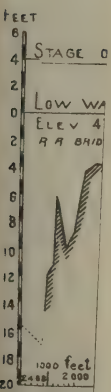
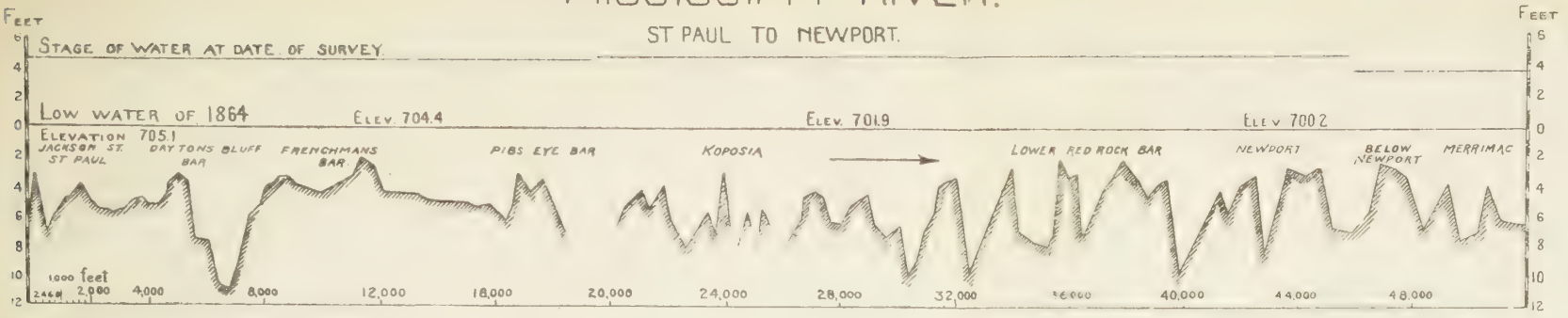

FEET
STAGE O
LOW WA
ELEV 4
R R BRID
1000 feet
2468 2000

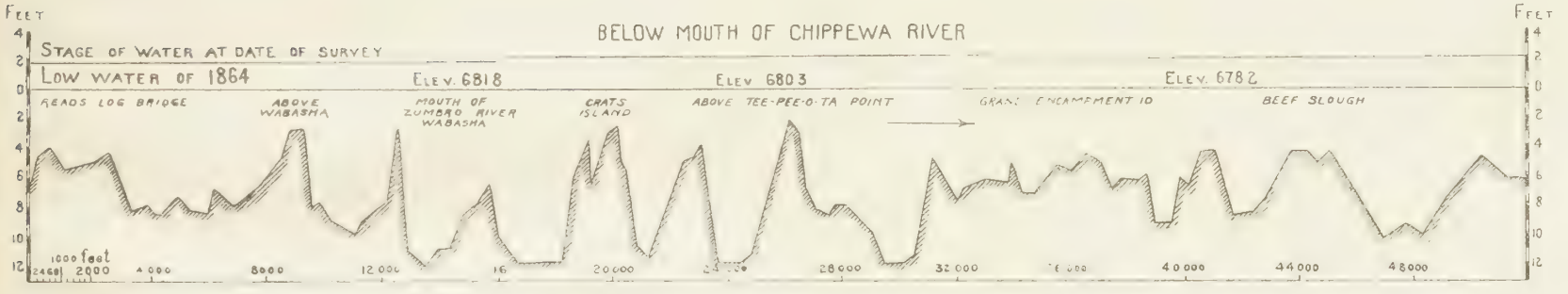
PLATE V

MISSISSIPPI RIVER.

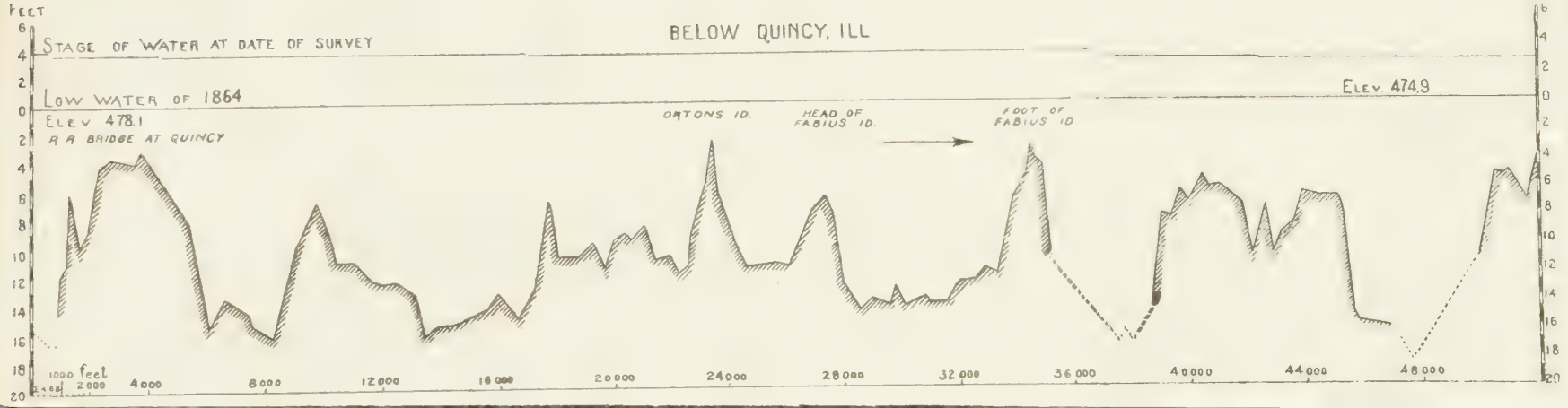
ST PAUL TO NEWPORT.



BELOW MOUTH OF CHIPPEWA RIVER



BELOW QUINCY, ILL



transported from the pool to the crossing exceeds the scour on the latter, and its height increases. As the river falls this process is reversed. (Starling on the Discharge of the Mississippi River, Vol XXXIV, Transactions of the American Society of Civil Engineers.)

In the Mississippi River below the mouth of the Missouri, changes in bar heights with river stage are large. The Board on Examination and Survey of the Mississippi River from St. Louis

to its mouth, with a view to obtaining a channel of 14-foot depth, reports that during ordinary floods the tendency of the bar to rise and fall, is at the rate of about 6 inches for a change of stage of 1 foot.

M. Girardin, in his report on the Rhône River, presented to the Sixth International Navigation Congress, estimates the ratio of change in bar height to variation of stage on that river as about one to five, and sums up the motion of sediment in a river as follows:

Natural water courses carry with their waters solid material.

The quantity of this material depends upon the resistance of the soil of the basin and the bed; it increases, other things being equal, with the volume of water and the slope.

The movement of the water is periodic; it passes from a weak to a strong discharge, and inversely, but once started it continues its motion to the sea. The movement of material follows the periods of the movement of the water, but in place of being continuous, is intermittent; its journey to the sea is affected by a series of étapes (a day's march with its stoppages for rest and refreshment).

These changes in the form of the river section have a marked effect on the distribution of its slope. During low stages the deep water in the pools and shoal water on the crossings produce gentle slopes through the former and steep slopes on the latter, and these changes are so marked that M. De Mas in his Navigation Interieur compares the water surface of a river to a staircase in which the surface of the pools corresponds to the steps and the surface of the crossings to the rise between them.

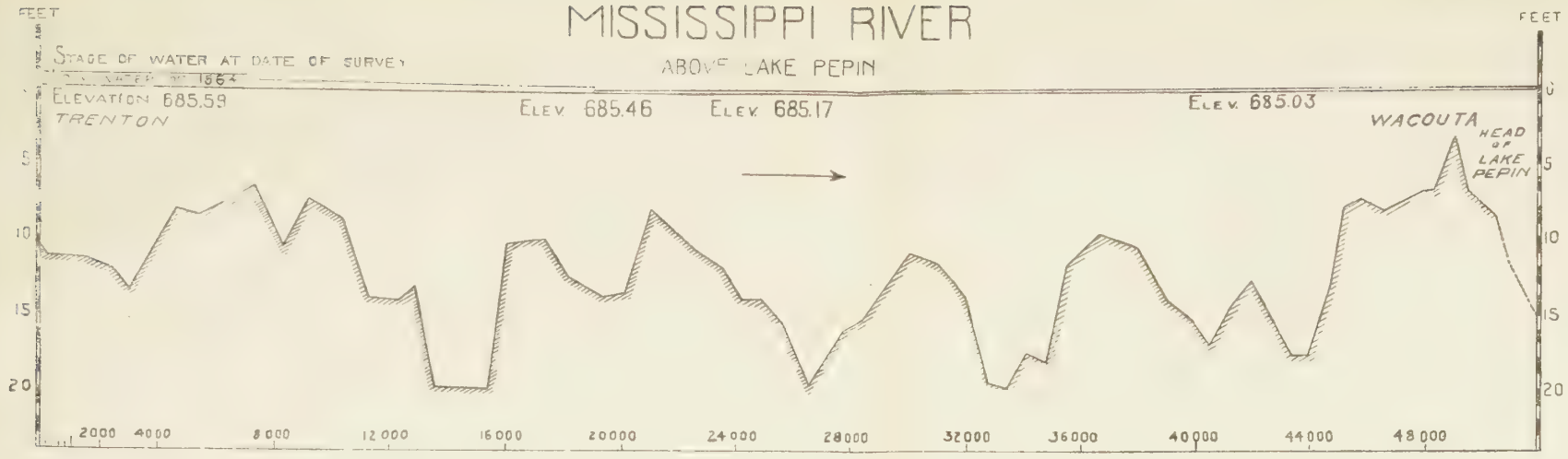
As the river rises the slopes on the crossings lengthen and become gentler. In the pools on the convex side of the bend the slope increases, but on the concave side the centrifugal force generated may produce very gentle slopes, so that at the lower end of a sharp bend there may be a steep slope between opposite banks. In the Mississippi River, differences of elevation of over 1 foot in the water surface on opposite sides of the river have been observed. If the bend is too deep for the discharge at a given stage to follow it, the flow across the point behind the towhead, as shown in Plate IV, Fig. 3, may cause a piling up of water at the lower end of the bend, so as to produce a negative slope through the bend, force the water to flow in an eddy upstream, and thus cause a fill in the bend instead of a scour.

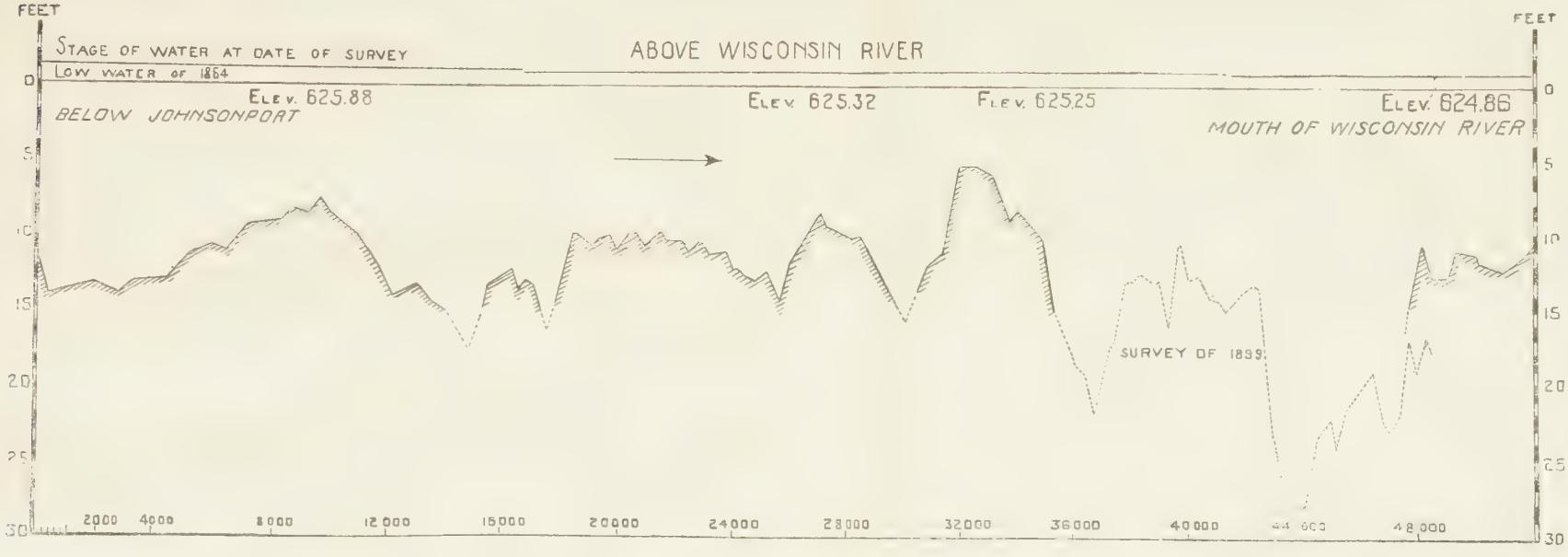
In addition to the change from the mean slope caused by pools and crossings, every stream which empties into the river affects its slope not only by its discharge but also by the amount of sediment it carries. If the tributary carries relatively more sediment than the main stream, the mean slope of the main river above the mouth of the tributary is diminished, and increased below. If it carries relatively less sediment, the reverse obtains, as is fully explained by Thomas and Watt in their work on the Improvement of Rivers.

In a channel with a fixed bed, such as a pipe, sewer or flume,

PLATE VII

MISSISSIPPI RIVER





it is possible to evolve a mathematical equation expressing the relation between the discharge, the area of cross section, and the slope or head, but from what has been explained above, it will be seen that the application of such an equation to a river will be difficult.

For example, take the well-known Chezy formula $V=c\sqrt{rs}$:— even in a smooth pipe, c, a coefficient which is a function of the roughness of the material which forms the channel, varies with the diameter of the pipe, and the head. When the inner surface of the pipe changes by rust or corrosion, there is also a marked change in the coefficient.

In a river the roughness of the bed varies with every cross section and every change of stage.

A sand wave moving along the bottom produces upward currents which appear at the surface as boils. In one bend the banks may be gravel, in another fine sand, having an entirely different coefficient of roughness; snags and boulders slide into the stream, causing whirls and eddies which tend to destroy the force which produces velocity.

In a pipe the area of cross section is uniform, but in a river the area of cross section, and therefore the hydraulic mean depth, at any particular locality, varies incessantly—a sand wave or the scouring of the bed may cause a variation of over 10 per cent in twenty-four hours. Nor are any two cross sections identical, nor do they vary by any law that can be expressed mathematically.

There is a similar lack of uniformity as to slopes.

During floods there may be some relation between the mean slope and the discharge, but during low water the controlling slopes are on the crossings, and at every crossing c, r and s differ, producing a different value for V.

In a channel with fixed bed and uniform cross section there can be expressed graphically a relation between discharge and stage, as shown on Plate VIII by the line AB. A similar line is frequently constructed for rivers, but it should be used with extreme care. It expresses the mean of numerous conflicting conditions.

On a rising river the slopes are greater than the mean slope, and on a falling river, less, and if there were no other conditions to be considered, the discharge curve instead of being the simple line AB, would conform to some loop such as CD, but as the river rises there is a raising of the bars which affects the area of cross

section at critical places, and if this cause was alone acting, the discharge curve would also be a loop. From both of these causes river slopes are continually changing and vary irregularly.

The building up of bars is not so rapid as the change of stage, but if for any reason the rise be checked, the building up of the bars may continue and produce great irregularities in the discharge. When to the above variations are added changes in cross section due to caving and sand waves, the line showing the relation of discharge to gage becomes instead of the curve AB the tangled skein shown in the plate.

The change of slope in a rising or falling river results from the reservoir capacity of the river bed, and the maximum discharge is frequently modified by the same cause. When the crest of a rise is passing down a river, there is a certain stage at every locality at which, if the river stands on the date the maximum discharge passes a station above it, there will result the same discharge at that locality. If it stands at a lower stage, a portion of the water is expended in filling the river bed, and the maximum discharge at the lower station will be diminished. If at a higher, the maximum discharge will be increased. The high stage at the lower stations usually results from the inflow of tributaries, and the results are obscured by the discharge of the tributary, but two crests may follow one another in rapid succession down the main stream, and the negative reservoir capacity above indicated then simply means that the second crest is overtaking the first, and appropriating to its discharge some of the water which had previously started down the river.

On the Mississippi River these results may be large. The crest of a rise of 50 feet at Cincinnati may pass Cairo within the river's banks, or threatening to destroy the levee line, dependent on the stage of the river at Cairo; and Humphreys and Abbot cite a case where the maximum discharge of the river was reduced 400,000 second feet in the passage of a flood wave from Columbus to Natchez.

For the same width of stream, the gentler the slope the greater is the reservoir capacity. Lake Pepin reduces a flood of about 20 feet at St. Paul to about 12 feet through the lake; and the maximum discharge at Carrollton is usually about 100,000 second feet less than that at Red River Landing, though a negative reservoir capacity is occasionally apparent between these stations.

From the above discussion it is apparent that a river flowing

PLATE VII

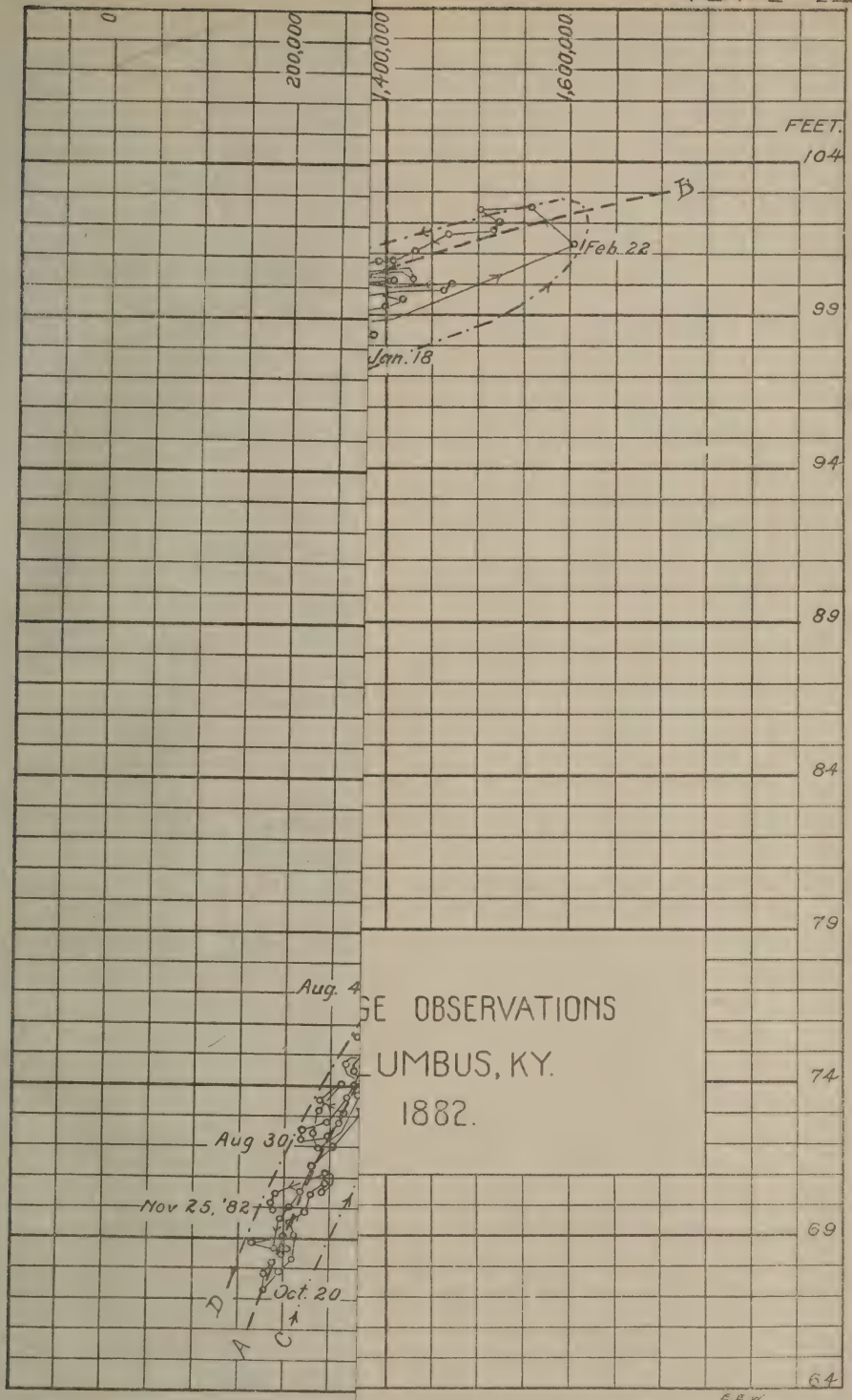
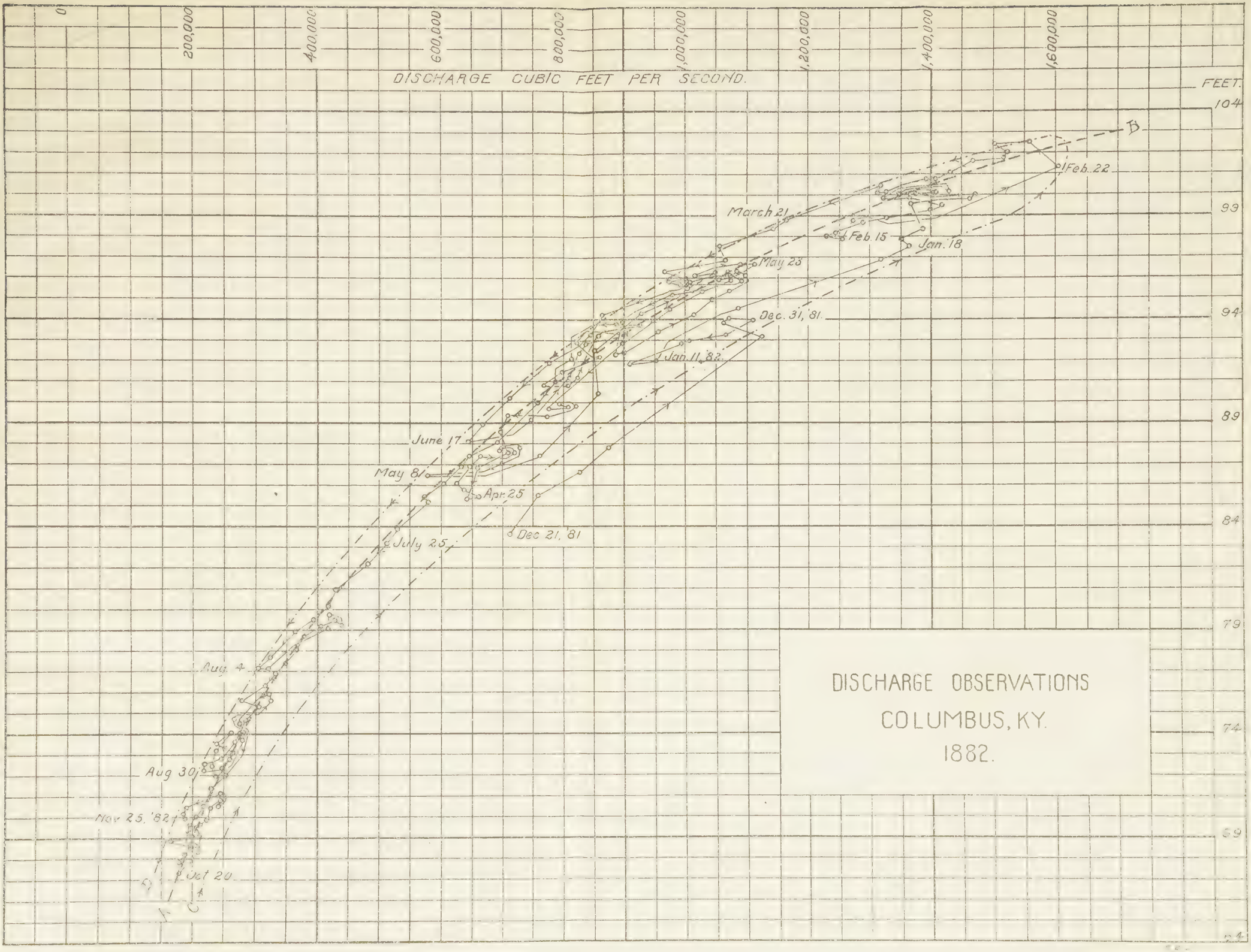

0
200,000
1,400,000
1,600,000
FEET.
104
99
94
89
84
79
74
69
64
B
Feb. 22
Jan. 18
Aug. 4
GE OBSERVATIONS
LUMBUS, KY.
1882.
Aug 30
Nov 25, '82
Oct. 20
D
A
C

PLATE VII


0
200,000
400,000
600,000
800,000
1,000,000
1,200,000
1,400,000
1,600,000
DISCHARGE CUBIC FEET PER SECOND.
FEET.
104
99
94
89
84
79
74
69
64
B
Feb. 22
March 21
Feb. 15
Jan. 18
May 23
Dec. 31, '81
Jan. 11, '82
June 17
May 8
Apr. 25
July 25
Dec 21, '81
Aug. 4
Aug 30
Nov 25, '82
Oct 20
DISCHARGE OBSERVATIONS
COLUMBUS, KY.
1882.

in an alluvial soil is constantly striving to adjust its bed to its discharge. The results of its efforts can best be determined by comparing stretches of river where the discharge varies. In Plates V and VI are thalweg depths through stretches where navigation was notoriously bad before the improvement of the river was undertaken, derived for the Upper Mississippi from the survey of 1879, and for Lake Providence Reach from that made by the Mississippi River Commission. Below St. Paul the extreme low water discharge is about 2,000 second feet; below Lake Pepin about twice as great; below Quincy about ten times; and through Lake Providence Reach over 50 times. It will be noted that the distance between crossings rapidly increases as the discharge increases, and also the depth in pools. Notwithstanding that the surveys of the reaches were made at different stages, it is a singular coincidence that on at least one crossing of every profile plotted, there is a depth of about 2 feet below the low-water line.

As the crests of bars rise and fall with the stage, the heights of crossings shown on the profiles would not obtain at low water. Nevertheless, the Reports of the Chief of Engineers indicate that at Lake Providence Reach the least depth on crossings has been as small at 4½ feet, below Quincy 2½ feet, at St. Paul, 2 feet. In other words, increasing the low-water discharge ten times has increased the depth on the shoalest crossing but 6 inches, increasing the low-water discharge fifty times has made a difference in the navigable depth of about 2½ feet. In Plate VII are the thalweg depths in stretches of the river where the navigation was exceptionally good before improvement. In these sections the mean slope is gentle and the caving slight.

Not only the slope but the shape of the bend above has a greater influence on the depth on the crossings than the volume of the discharge.

These observations conclusively prove that a flood does not flush out a river, leveling off its bars, like a rapid discharge acts on the deposits in a sewer, as has been claimed by some writers, but on the contrary, that the action of the flood is to increase the difference in depth on pool and crossing, while that of a low stage is to diminish it.

This, however, affords no evidence that an increase in flood discharge causes a general rising of the river bed. While the depth on crossings increases less rapidly with the discharge than many

writers have claimed and ordinary hydraulic formulas would indicate, there is still an appreciable increase.

In some remarks I made at Memphis in September, 1912, I called attention to the conditions under which the discharge of a stream would raise or lower its bed. These do not depend on the amount of the discharge so much as on the slope of the land over which the stream flows. If a stream flows down a steep hill into a valley, it washes the material from the crest of the hill and ordinarily forms a cone of deposit at its entrance into the valley. When the stream is limited in width by levees, this cone is limited to the space between them, and there is at that locality a rapid rise of the stream's bed, as is illustrated by the flow of Bear Creek into the Mississippi a short distance above Quincy, Ill.

If a dam is constructed across a stream, there will also be a deposit above it raising the bed. If the dam for any cause be removed, the stream will tend to remove this deposit. There are temporary variations in slope in every river, frequently caused by cut-offs, on which the discharge has a similar effect, and the greater the discharge the more rapidly this effect is produced. An interesting illustration of the above is afforded by the portion of the Red River shown in profile in Plate IX.

This portion of the river, whose waters at mean stages attain about the same degree of saturation with sediment as the Missouri, was obstructed by a raft, which reduced its discharge and caused a material raising of its bed. The water which was unable to force its way down the main stream, overflowed its banks and cut numerous auxiliary channels through the alluvial valley. This raft was removed from 1872 to 1880, and since that date the water has been confined to a single channel by the closure of chutes and the construction of levees. By these means a large increase in the discharge of the main river has been created.

There has resulted therefrom a large increase in the area of cross section and a lowering of the bed in some cases amounting to about 30 feet, but with this violent change in regimen there is still found the same alternation of pool and crossing as existed before the improvement was undertaken, and a comparison of soundings taken in 1894 and 1910 indicates that the net result of all these changes as far as concerns low-water depths on crossings, is an increase of from 2 to 4 inches.

In the portion of the river below Shreveport, shown in profile on Plate X, the rafts were removed prior to the Civil War, so that the river had an opportunity to adjust its slopes to the changed

PLATE IX.

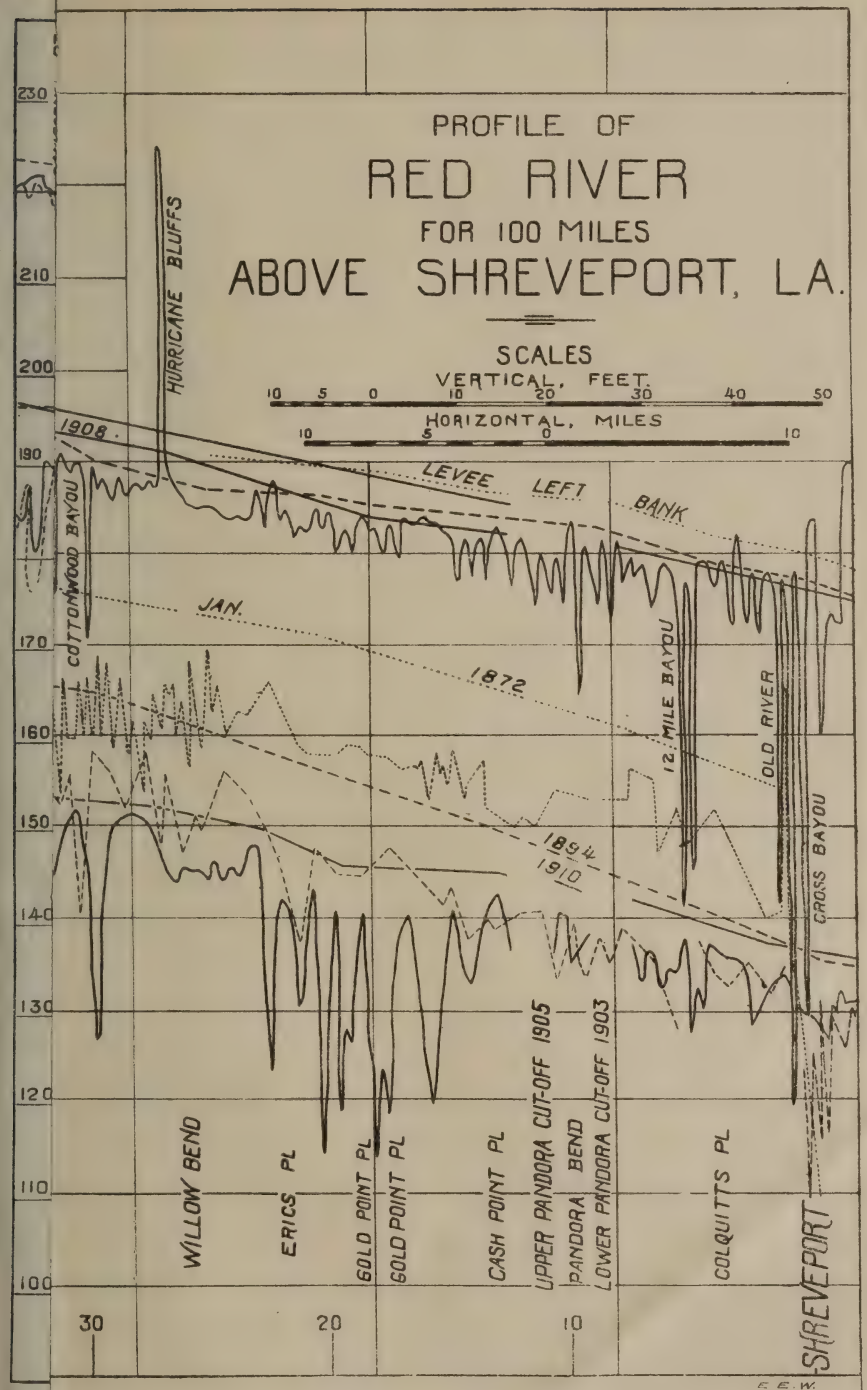
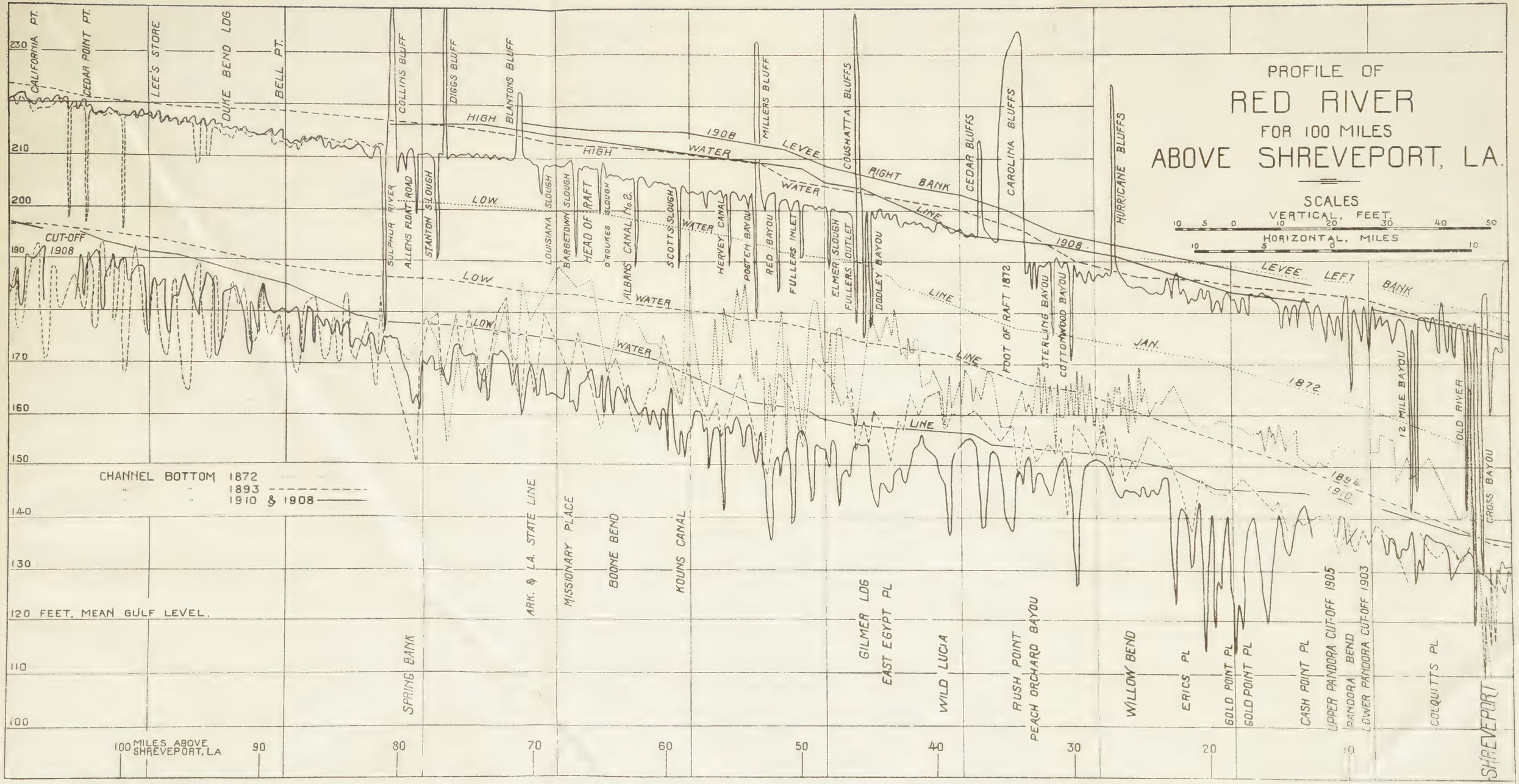

PROFILE OF
RED RIVER
FOR 100 MILES
ABOVE SHREVEPORT, LA.
SCALES
VERTICAL, FEET.
10 5 0 10 20 30 40 50
HORIZONTAL, MILES
10 5 0 10
HURRICANE BLUFFS
1908.
LEVEE
LEFT
BANK
JAN.
1872
1894
1910
COTTONWOOD BAYOU
12 MILE BAYOU
OLD RIVER
GROSS BAYOU
230
210
200
190
170
160
150
140
130
120
110
100
30
20
10
WILLOW BEND
ERICS PL
GOLD POINT PL
GOLD POINT PL
CASH POINT PL
UPPER PANDORA CUT-OFF 1905
PANDORA BEND
LOWER PANDORA CUT-OFF 1903
COLQUITTS PL
SHREVEPORT

PLATE IX.


PROFILE OF
RED RIVER
FOR 100 MILES
ABOVE SHREVEPORT, LA.
SCALES
VERTICAL, FEET.
10 5 0 10 20 30 40 50
HORIZONTAL, MILES
10 5 0 10
CALIFORNIA PT.
CEDAR POINT PT.
LEE'S STORE
DUKE BEND LOG
BELL PT.
COLLINS BLUFF
DIGGS BLUFF
BLANTONS BLUFF
MILLERS BLUFF
COUSHATTA BLUFFS
CEDAR BLUFFS
CAROLINA BLUFFS
HURRICANE BLUFFS
SULPHUR RIVER
ALLENS FLOAT ROAD
STANTON SLOUGH
LOUISIANA SLOUGH
BARGETOWN SLOUGH
HEAD OF RAFT
O'ROURKES SLOUGH
ALBANS CANAL No. 2.
SCOTTS SLOUGH
HERVEY CANAL
POSTEN BAYOU
RED BAYOU
FULLERS INLET
ELMER SLOUGH
FULLERS OUTLET
DUDLEY BAYOU
STERLING BAYOU
COTTONWOOD BAYOU
12 MILE BAYOU
OLD RIVER
CROSS BAYOU
HIGH WATER LINE 1908
LOW WATER LINE 1908
RIGHT BANK LEVEE
LEFT BANK LEVEE
FOOT OF RAFT 1872
JAN. 1872
CUT-OFF 1908
CHANNEL BOTTOM 1872
1893
1910 & 1908
120 FEET, MEAN GULF LEVEL.
230
210
200
190
170
160
150
140
130
110
100
100 MILES ABOVE SHREVEPORT, LA
90
80
70
60
50
40
30
20
10
SPRING BANK
ARK. & LA. STATE LINE
MISSIONARY PLACE
BOONE BEND
MOUNS CANAL
GILMER LOG
EAST EGYPT PL
WILD LUCIA
RUSH POINT
PEACH ORCHARD BAYOU
WILLOW BEND
ERICS PL
GOLD POINT PL
GOLD POINT PL
CASH POINT PL
UPPER PANDORA CUT-OFF 1905
PANDORA BEND
LOWER PANDORA CUT-OFF 1903
COLQUITTS PL
SHREVEPORT

conditions before the first survey was made. Early records indicate that this part of the river had originally as many outlets as the raft region, but these outlets had become filled with sediment to a higher stage.

The removal of the raft in the upper river, however, largely increased the discharge, and the river has markedly increased its width and has attempted to adjust the curvature of its bends to the increased discharge, with the result that there have been numerous cut-offs, some artificial, which have reduced the length of the river between Shreveport and Alexandria (formerly 218 miles) about 26 miles. There has also been a lowering of the low-water surface of about 5 feet at the upper end of the reach, and a raising of the river bed of about 3½ feet at the lower end.

Humphreys and Abbot, in their Physics and Hydraulics of the Mississippi River, state that the normal effect of a cut-off is to raise the water level just below it by an amount equal to half the fall of the river in a straight portion equal in length to the shortening of the channel, and to lower it just above by an equal amount plus the fall required to overcome the resistance of the bend.

The above will explain the changes of slope observed, but they could also be accounted for by the movement of sand from the section above Shreveport, as observations on the Sacramento River indicate that the crest of the sand waves caused by hydraulic mining and which are composed of heavier material than those on the Red River, move down that river at the rate of about 4 miles per year.

The change of regimen in this section of the river has also had but slight effect on the low-water depths on the crossings.

In applying these principles to spillways in the vicinity of New Orleans, it is to be noted that the conditions governing the flow of water through a spillway differ materially from those through a crevasse, or an outlet. When a crevasse occurs there is a violent local change in the regimen of the river. The flow through the crevasse at first suddenly accelerates velocities above it and retards them below.

A sudden increase in velocity is conducive to scour, and the formation of sand waves. A large amount of sediment is also placed *intermittently* in suspension. The retardation of velocities below the crevasse tends to cause a deposit, both of the sand waves and of the material intermittently in suspension.

But, due to the frictional resistance of the river bed, this scour and fill is confined to a comparatively short distance, and as soon as the reservoir capacity of the river adjusts itself to the new conditions, abnormal velocities both above and below the crevasse cease.

As soon as the flood subsides to a bank full stage, there is a tendency for the river to return to its normal form, filling the channels scoured above the crevasse and removing the deposits below.

When the river rises over the crest of a spillway no sudden change will occur either in the reservoir capacity, slopes, or velocity. Above it the slope and velocity will gradually increase; below they will also increase, but the increment will be less, and, due to the proximity of New Orleans to the Gulf and the absence of any tributaries to the river below that city, the changes in these increments will not be as great as are liable to occur at localities farther up the river from the sudden inflow from tributaries, particularly at the mouths of the White and Arkansas rivers.

A spillway also differs from an outlet in that the outlet is discharging at all stages, and diminishes the flow at low water, when the crossings are being scoured, and all the water that can be utilized should be employed to increase such action. The spillway abstracts water from the river only at high stages when the crossings are filling. It will affect the amount of oscillation of the bars more than the ultimate depth of the crossing at low water.

At New Orleans river slopes are gentle, and the depths on crossings at low water exceed 75 feet; so that as far as navigation is concerned even for the largest ocean vessels, a considerable shoaling of the crossings could be permitted.

If crevasses had not occurred in the levee line, 400,000 second feet could have been abstracted from the river by spillways during the flood of 1912, and the discharge at New Orleans would still have exceeded that of 1882, when ample depths existed in the river from that city to the Passes. Moreover, the increase in discharge during floods since that date has had no appreciable effect on the area of cross section of this portion of the river.

But in determining the effect of the spillways on flood heights, the employment of a mean discharge curve will lead to erroneous results. For, though due to the limited degree of saturation of the water, considerable variations in velocity may be permitted with-

PLATE X

PROFILE OF RED RIVER FOR 100 MILES BELOW SHREVEPORT, LA

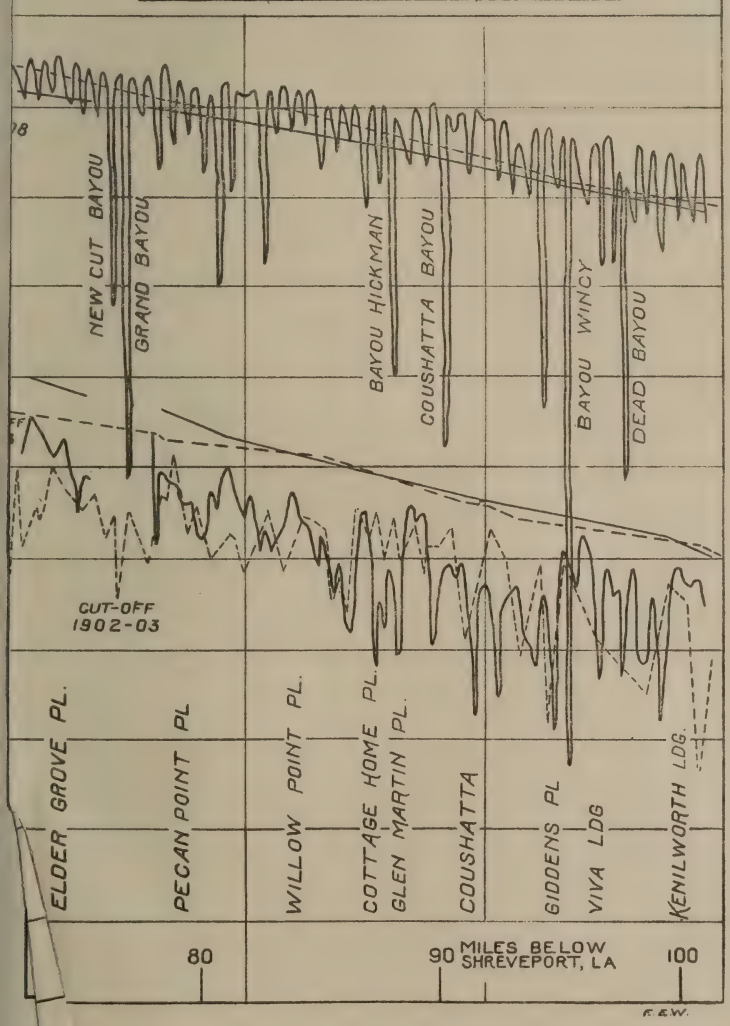
SCALES

VERTICAL, FEET

10 5 0 10 20 30 40 50

HORIZONTAL, MILES

10 5 0 10



F.E.W.

PLATE X

PROFILE OF RED RIVER FOR 100 MILES BELOW SHREVEPORT, LA

SCALES
VERTICAL, FEET
10 5 0 10 20 30 40 50
HORIZONTAL, MILES
10 5 0 10

CHANNEL BOTTOM 1893 ----------
" " 1908 & 1906 ——

1908 LEVEE LEFT BANK
1908 LEVEE RIGHT BANK
HIGH WATER LINE 1894
HIGH WATER LINE 1908
LOW WATER LINE 1910
LOW WATER LINE 1894

180
170
160
150
140
130
120
110
100
90
80
70
60 FEET, MEAN GULF LEVEL
50

12 MILE BAYOU
OLD RIVER
CROSS BAYOU
BAYOU PIERRE
MACKS BAYOU
SHREVES CUT-OFF
LATTIERS CUT-OFF
TONES BAYOU
WATERLOO CUT-OFF
YOUNGS PT. CUT-OFF 1890
HINOCK BAYOU
SWIFT CHUTE BAYOU
BAYOU LA CHUTE
LOGGY BAYOU
NEW CUT BAYOU
GRAND BAYOU
BAYOU HICKMAN
COUSHATTA BAYOU
BAYOU WINCY
DEAD BAYOU

CUT-OFF 1902-03
CUT-OFF 1908
CUT-OFF 1902-03
CUT-OFF 1902-03
CUT-OFF 1902-03
CUT-OFF 1907
CUT-OFF 1908
CUT-OFF 1895
CUT-OFF 1905
CUT-OFF 1902-03
CUT-OFF 1903
CUT-OFF 1903
CUT-OFF 1902-03

COLQUITT'S PL.
SHREVEPORT
CHALK LEVEL SHOAL
NICHOLSON PL.
CURTIS PL
SUNNY POINT PL.
CHILDERS PL.
LOTUS LDG.
PASCAGOULA PL.
KNOX POINT PL.
CASPIANA PL.
CAMPOBELLO PL.
CROSS KEYS PL.
HOWARD P.O.
RAINBOW BEND PL.
NELSON PL
ELDER GROVE PL.
PECAN POINT PL
WILLOW POINT PL.
COTTAGE HOME PL.
GLEN MARTIN PL.
COUSHATTA
GIDDENS PL
VIVA LDG
KENILWORTH LDG.

0 10 20 30 40 50 60 70 80 90 100 MILES BELOW SHREVEPORT, LA

C.E.W.

out an appreciable deposition of sediment, these variations will have a material effect on flood heights. It will be more accurate to construct the loop CD on Plate VIII, using the portion of the curve for a rising river in discussing the conditions above the spillway, and that for a falling river below it. For high stages there is a difference of about 100,000 second feet between the two branches of the curve.

The effect of the flow of water through the spillway itself is not so satisfactory. As stated above, the waters of the Mississippi carry a variable amount of sediment, dependent on the source of the flood. If principally derived from the Missouri, Arkansas and Red rivers, every 1,000 cubic feet of water flowing may contain over 1 cubic foot of sediment, and if the spillways were permitted to discharge 400,000 second feet of water, they might also discharge over 1,280,000 cubic yards of sediment per day. The lakes into which they must empty are shallow, and there are no lateral currents to carry the sediment long distances, so that it must be deposited in a fan-shaped bar around the outlet, similar to that which now forms in the Gulf of Mexico at the river's mouth. Unless the bar is annually dredged, it will react on the slope of the spillway and cause it to fill, reducing its discharge. It is a similar action that has prevented the large crevasses on the lower river, such as the Morganza and Bonne Carré, from developing into outlets.

The laws which have caused the formation of the channel have existed for geological eons, long before levees were thought of, and while the law that a river flowing in an alluvial soil will adjust itself to its discharge is immutable, it will strive to adjust itself to any irregularity that may exist in the levee line by reason of a spillway by reducing the area of the main channel very slowly and by closing up the spillway rapidly.

Moreover, the outlets to the lakes have adjusted themselves to their existing discharge, and if this discharge be greatly augmented by the inflow from a spillway, the surface of the lake must rise. Crevasses have caused a rise in Lake Pontchartrain of over 3 feet.

In conclusion it is submitted that while the construction of a spillway would not injuriously affect navigation, and would materially reduce flood heights, to maintain it after construction would require such an enormous amount of dredging as to render the entire project impracticable, without even taking into consideration the necessity of leveeing Lake Pontchartrain.

Maj. Gen. James Birdseye McPherson.*

BY

Maj. AMOS A. FRIES,
Corps of Engineers.

Gen. James B. McPherson was born at Clyde, Sandusky County, Ohio, November 14, 1828. His father, William, was of Scotch-Irish descent and his mother, whose maiden name was Cynthia Russell, was of English descent and a native of Massachusetts. William McPherson died in 1841, leaving James B., then thirteen years of age, to aid his mother to make a living for themselves and three children younger than himself. As a boy James showed alertness of mind, and after the death of his father easily obtained employment with Mr. Robert Smith, postmaster and storekeeper at Green Springs near Clyde. He remained in this employment for six years, meanwhile attending school in the winter and utilizing his spare time in study at home. He seems to have had from early childhood the faculty of making friends; it was this quality, probably, that led to his being promised an appointment to the Military Academy at West Point when he was nineteen years of age. Upon receiving this promise, he gave up his clerkship under Mr. Smith and took a short course at Norwalk Academy preparatory to entering the Military Academy.

He entered West Point in 1849 at the age of 20, and was graduated four years later number one in his class. Very little is known of his career at West Point, but it is evident that his quickness of mind as a boy was not overrated and that the course at West Point merely developed a natural brilliancy of mind enabling him, in spite of his meager educational opportunities be-

*Literature on the life of General McPherson is very scarce. The notes concerning his life prior to entering West Point are taken mainly from the National Cyclopedia of American Biography, Vol. IV, while that concerning his Army career is principally from Cullom's Biographical Register of Graduates of the United States Military Academy, Vol. II.

fore entering West Point, to take his position at the head of his class.

His class standing and other qualities caused him to be retained at West Point on graduation as instructor in engineering until 1854, when he was assigned to duty in New York as assistant on the construction of the defenses of that harbor and the improvement of the Hudson River. In 1857 he was placed in charge of the construction of Fort Delaware, going from there to the defenses of Alcatraz Island, San Francisco Harbor. He was promoted to first lieutenant in 1858 and, in 1861, was ordered to Massachusetts, where he was engaged in organizing engineer troops for duty in the Civil War. In August of that year he was promoted to be captain of Engineers, and in November was chosen as an aide-de-camp by General Halleck. In addition to his duties as aide-de-camp he was Assistant Engineer of the Department of the Missouri with the rank of lieutenant colonel.

In 1862, beginning with his promotion to colonel in May, he was progressively advanced to the grades of brigadier general, U. S. Volunteers, and major general, U. S. Volunteers. When active operations began along the Mississippi River he was made chief engineer on the staff of General Grant, and from that time until General Grant was transferred to the East in command of all the Union forces General McPherson was with Grant in every battle, always making himself conspicuous by his skill and daring both in reconnaissance and in handling troops. He seems always to have acted as chief engineer, notwithstanding he very soon rose to command an army corps.

To enumerate all the battles in which General McPherson took an important and generally brilliant part would be simply to enumerate all the battles in the Mississippi Valley under General Grant, beginning with Fort Donelson. No greater eulogy can be written of General McPherson's work than that by General Grant himself after the capture of Vicksburg, when he recommended McPherson to be appointed a brigadier general in the Regular Army.

McPherson has been with me in every battle since the commencement of the Rebellion, except Belmont. At Forts Henry and Donelson, Shiloh and the Siege of Corinth, as a Staff Officer and Engineer his services were conspicuous and highly meritorious. At the second Battle of Corinth his skill as a soldier was displayed in successfully carrying reinforcements to the besieged garrison when the enemy was between him and the point

to be reached. In the advance through Central Mississippi, General McPherson commanded one wing of the army with all the ability possible to show, he having the lead in the advance, and the rear retiring.

In the Campaign and Siege terminating with the fall of Vicksburg, General McPherson has filled a conspicuous part. At the Battle of Port Gibson, it was under his direction that the enemy was driven, late in the afternoon from a position they had succeeded in holding all day against an obstinate attack. His Corps, the advance always under his immediate eye, were pioneers in the movement from Port Gibson to Hawkinson's Ferry. From the North Fork of the Bayou Pierre to Black River it was a constant skirmish, the whole skillfully managed. The enemy was so closely pressed as to be unable to destroy their bridge of boats after them. From Hawkinson's Ferry to Jackson, the Seventeenth Army Corps marched on roads not traveled by other troops, fighting the entire Battle of Raymond alone, and the bulk of Johnston's Army was fought by this Corps, entirely under the management of General McPherson. At Champion Hill, the Seventeenth Corps and General McPherson were conspicuous. All that could be termed a battle there was fought by the divisions of General McPherson's Corps and General Hovey's Division of the Thirteenth Corps. In the assault of the 22d of May on the fortifications of Vicksburg, and during the entire Siege, General McPherson and his command took unfading laurels. He is one of the ablest engineers, and most skilled generals. I would respectfully but urgently recommend his promotion to the position of Brigadier General in the Regular Army.

General McPherson achieved the remarkable distinction of being "awarded a medal of honor by the officers of his corps for the gallant manner in which he led them during the campaign and Siege of Vicksburg."

At the surrender of Vicksburg General McPherson was chosen one of the Commissioners to arrange the terms of capitulation, and after the surrender was assigned to the command of the city and district of Vicksburg, where he remained until March 26, 1864. During this period he participated in Sherman's expedition to Meridian, Miss., which resulted in the destruction and capture of large quantities of supplies and other resources of the Confederacy. In addition, as commander of the District of Vicksburg, he directed important raids into Mississippi and Louisiana.

When General Grant left the Western Army for Washington he wrote to General Sherman as follows.

I want to express my thanks to you and to McPherson as the men to whom, above all others, I feel indebted for whatever I have had of success.

Upon Grant's departure, Sherman was appointed his successor, while McPherson was given command of the Army of the Tennessee. Colonel Chesney, of the British Royal Engineers, in writing of Generals Grant, Sherman and McPherson, said, "It is hard to say whether Grant leant more on the calm courage and unfailing resources of Sherman or the subtle genius and daring spirit of the lamented McPherson, a soldier of the very highest promise."

The Army of the Tennessee, which had been reduced to about twenty-three thousand men, was reorganized at Huntsville, Ala., by McPherson, and on May 6, 1864, entered as a part of Sherman's Army upon the Atlanta Campaign. This campaign from its start early in May until the fall of Atlanta, August 1, was a nearly continuous battle. As is known to every student of history, Gen. Joseph E. Johnston opposed Sherman's Army until just before the fall of Atlanta, when he was succeeded by Hood. Johnston, realizing that his forces were inferior to those of Sherman, chose to fight on the defensive, though never losing an opportunity to make an able counterstroke if such afforded chances of success. However, every time he fell back, which was always for only a few miles, he strongly entrenched. These entrenchments proved so well built and their location so well chosen that after a few attempts Sherman's Army practically gave up the hope of taking them by assault and proceeded only by outflanking them. In all of these moves Sherman placed the greatest confidence in McPherson and usually gave to him the most difficult task of all.

When almost within rifle shot of Atlanta and during almost the last important sortie or counterstroke attempted by the Confederates, McPherson was killed while forming his lines to meet an attack on his flank and rear. At the moment of his death the attack was practically at a standstill and was soon thereafter repulsed. He had scattered his aides with various messages and was riding along a trail leading from one part of the lines to another, accompanied only by an orderly, when he ran into the Confederate skirmish line. Whether or not he was recognized by the Confederates as General McPherson is questionable, but undoubtedly they saw that he was an officer of high rank and called upon him to surrender. Here his habit of ever keeping his duty to his country and to his command first in his thoughts led to his nearly instant death. He unquestionably realized that he

was courting death when, refusing to surrender, he raised his hat in apparent salute and turned swiftly to gallop into the woods at one side. At once a volley from the skirmish line brought him from his horse with wounds from which he died soon afterward.

To General Sherman McPherson's death was a terrible blow, and that night he said to a staff officer while riding from the scene of the battle: "The Army and the Country have sustained a great loss by the death of McPherson. I had expected him to finish the war. Grant and I are likely to be killed or set aside after some failure to meet public expectation and McPherson would have come into chief command at the right time to end the war. He had no enemies." We find another tribute in a letter which General Grant wrote to McPherson's grandmother where he says in part, "He formed for sometime one of my military family. I knew him well. To know him was but to love him. It may be some consolation to you, his aged grandmother, to know that every officer and every soldier who served under your grandson felt the highest reverence for his patriotism, his zeal, his great, almost unequaled, ability, his amiability, and all the manly virtues that can adorn a commander."

Gatun River Bascule Bridge, Panama Railroad.*

BY

Mr. P. L. KAUFMAN.

The relocation and consequent reconstruction of the Panama Railroad presented some very unique problems in the field of railroad engineering.

The Panama Railroad has played an important part in the construction of the great canal and will continue to serve as a necessary and useful adjunct to its operation in the future. Very little, however, in comparison with the canal itself, has been written concerning the engineering and constructive features relative to the railroad. In ordinary railroad work elsewhere, the engineer changes the topography of the land to suit primarily the needs of the railroad itself but in the case of the Panama Railroad, the construction of the canal wrought such radical changes in the natural conditions that the engineers were confronted with many unusual conditions, but at the same time the construction of the canal, in some ways, assisted the engineers in their work in planning the railway.

The total length of the relocated or new line is 40 miles, extending from Mindi near the Atlantic terminal to Corozal near the Pacific terminal, and represents an expenditure of $8,870,000 or about $221,500 per mile. Included in this improvement was the construction of the Gatun River Bridge.

Since the waters of Gatun Lake were raised considerably, due to the construction of the Gatun Dam, what was formerly known as Gatun River became the upper arm of the lake. Anticipating this change in conditions, a site was selected for the bridge (or more correctly speaking, for the river), which would secure the most satisfactory and economical foundation for the bridge piers. The elevation of the old river bed was 17, which would have re-

*Contributed by Mr. P. L. Kaufman of the Strauss Bascule Bridge Company of Chicago, Ill.

quired piers about 85 feet high, whereas the site selected required piers 58 feet high. This, however, necessitated the excavation of a new river channel from the west side to provide a final depth of water of 45 feet. In other words, the unique condition presented itself of building a bridge and then building the river to suit the bridge's convenience instead of the reverse, as is usually the case. Fig. 1 is a view of the site, taken June, 1910. This illustration shows the temporary embankment and the final location of the center line of the railroad and, in the foreground, the final location of the Gatun River bridge; the temporary crossing of the Gatun River is shown in the center background.

Fig. 2 is a view taken also in June, 1910, showing at close range the temporary crossing of the river, referred to in the previous paragraph. One of the old Panama Railroad girder spans, taken out of the Barbacoas Bridge at San Pablo, was utilized for this temporary crossing. The conditions encountered thus presented great constructive advantages which can readily be realized by reference to Fig. 3, which indicates the piers practically completed. This view was photographed in March, 1911, and from the opposite side and in the opposite direction from that shown in Fig. 1. The temporary track is indicated on the extreme left, which was used in constructing the first deck at Elevation 60 of the fill for the new line, and the trestle for the second deck at Elevation 92 is indicated on the right. This trestle was also utilized in placing the new girders on the piers. These piers were built without having to contend with water and practically no excavation was necessary. Fig. 4 is a general drawing of the bridge including a profile of the river bed, and showing the elevation of the various water stages. Figs. 5, 6 and 7* show views of the completed structure.

The bridge is located near the south end of Gatun Valley near Monte Lirio station, the central span being a bascule to permit access by vessels to the upper arm of Gatun Lake and River, which, after the completion of the canal, has become navigable and will assist in developing the territory contiguous to the upper arm of the lake and river.

Referring to Fig. 4, it will be seen that the structure consists of three plate girder spans of practically the same length, of which the bascule is 106 feet 7 inches. These three girder spans were already on hand, it being the original intention of the Isth-

*Fig. 7 not printed.

mian Canal Commission not to provide for navigation above the bridge and later, when it was decided to permit navigation above the bridge, it therefore remained to convert the central span into a movable span with the least possible changes and alterations to the girders as fabricated.

The type of bascule found suitable for these conditions was the Overhead Counterweight Type of the Strauss design.

Fig. 8 shows in detail the method employed to make the span operative, including the arrangement of the operating machinery.

As will be noted, two short longitudinal lifting trusses are

Fig. 1. Relocation of Panama R. R. Showing temporary embankment and the final location of center line of railroad. Final location of bridge indicated in foreground.

connected by means of vertical diaphragms, at the end panel points, to the main girders. These trusses are mounted, by means of trunnions secured by fixed collar bearings to their top chords, in symmetrical trunnion bearings, whose centers are coincident with the center line of the pier. The outside trunnion bearing is secured to a structural steel frame anchored to the pier, which also carries the column for the counterweight link above. The inside trunnion bearing is secured to the top flange of the approach girder, which latter is reinforced with stiffener angles between the top and bottom flanges under this bearing.

This approach girder, as well as the bascule girder, is supported on a short structural steel shoe anchored to the masonry. At the rear extremity of the lifting trusses counterweight trunnions are secured to the top chords, as indicated in Fig. 8.

These counterweight trunnions support the concrete counterweight above by means of structural steel columns, or legs, which are provided with bearings secured to them at their bases in which the counterweight trunnions are free to turn. The counterweight is guided and maintained in a vertical position by means of the well-known parallel system, typical of many of the Strauss designs, and is apparent by reference to the illustrations.

The center of main trunnion about which the trusses rotate is the center of gravity of the entire moving structure. This point, the counterweight trunnion and the combined center of gravity of the main girders and lifting trusses lie in a straight line; the weights of counterweight and moving leaf are inversely proportionate to their leverages about the main trunnion; these conditions and the parallel movement of the counterweight maintain the structure in equilibrium in all positions.

While the structure is balanced, mechanical power is required to overcome the inertia, wind and friction, in opening and closing the bridge. Both electric and emergency hand operation are provided for this purpose.

The operating machinery is located on top of the trunnion pier and in two symmetrical units outside of each lifting truss and is suitably housed. The main operating pinions are keyed to a shaft, which extends across the bridge below the trusses. These operating pinions engage circular racks secured to the bottom chords of the trusses, whose centers of rotation are the main trunnions. Each operating pinion is actuated by a 22 horsepower alternating current motor (710 r. p. m. 25 cycle, 220 volt, three-phase circuit) connected to the operating pinion through four gear reductions, which are arranged as indicated in Fig. 8. No equalizing device is required, since the operating pinions are keyed to one shaft. This shaft turns in bearings secured to the base of the outside trunnion posts and the structural steel shoe under the ends of the main girders, which serve as very rigid supports; this method of support increases the efficiency of the machinery to a marked degree.

The motor is controlled from an operator's house of reinforced concrete located at the end of the bridge, which is shown in Fig.

6. The electrical control is interlocked with the automatic signal system of the railroad, to guard against trains proceeding toward the bridge at the times of operation.

The hand operating devices are two in number and are located on top of the machinery enclosures on either side; each consists primarily of a windlass with double hand cranks, which cranks are connected to the operating machinery by two sets of chain sprocket wheels as indicated in Fig. 8.

The sprocket wheel on the motor shaft is not keyed thereto, but is provided with a clutch which is operated by means of a

Fig. 2. Relocation of Panama Railroad. Showing temporary crossing of the Gatun River, at Monte Lirio, to accommodate traffic while building the permanent bridge.

lever from the operator's platform, and when thrown in becomes keyed to the motor shaft through the agency of the clutch. When the clutch is withdrawn, the motor shaft turns free in the hub of this sprocket wheel and the hand-operating device remains stationary.

For hand operation a foot brake is also provided (see Fig. 8), operated from one side only, but the brake mechanisms on either side are connected by means of a cross rocking shaft. One 100-pound weight operates each brake, the leverage of which can be adjusted as indicated. The motors are provided with dead-weight

solenoid brakes, springs not being desirable on account of the corrosive influence of the climate.

Since the plate girders are assumed to carry the live load as in an ordinary fixed structure, a positive lock is required to force them down tight on the shoes. The locking mechanism which is located on the front end of the moving leaf, as indicated in Fig. 8, has proven positive and definite in action. The locking mechanism is driven by a 3 horsepower alternating current motor (220 volt, 3 phase, 25 cycles) and is electrically interlocked with the track signal and leaf motors to insure their operation in correct sequence.

Extracts from the specifications, stipulated by the Isthmian Canal Commission governing the selection of a design, are reprinted herewith as follows:

* * * * * * *

5. All bids must be on a type of lift bridge which is adaptable without alteration of the substructure as built and which will make use of the three existing plate girder spans with the minimum possible alterations. . . .

6. Special attention must be given to any change of stress in the existing girders from those for which they were originally designed, and the ultimate stresses must be in strict accordance with the provisions of these specifications. In case the dead load of the movable span comes upon the adjacent approach span, either when the bridge is closed or during operation, this span must be figured for the weight of the span itself and the weight of the movable span and counterweight, proper reduction being made in the unit stresses as specified below to take care of impact, vibration, machinery effect, etc.

7. Design shall be of such character that all parts, including operating devices, are rigid and positive and free from material deformation and indeterminate stresses. No design will be considered having the counterweight immersed or dipping below extreme high water.

8. It is desired that the center of gravity of the lifting span, open or closed, be kept close to the base of rail, and that the center of pressure of the wind force on the opened span should be as low as possible. Preference will be given to designs in which the operating machinery is located on the stationary part of the structure.

9. In effecting the connection of the present plate girder to the lifting parts, both the top and bottom flanges of the plate girders must be thoroughly connected to the lifting mechanism and full provision must be made for the transfer of the stresses under all conditions.

10. In order to receive due consideration, each bid must be accompanied by full and complete description, together with drawings and stress sheets as follows:

(*A*) Complete stress sheets showing assumed live loads, weight of floor, weight of steel, the panel concentrations of all dead loads; live impact and dead-load stresses separately and combined; the section; gross and net areas of each member; clearances when the bridge is opened and closed, maximum reaction on the piers, character of the counterweight and assumed weight per cubic foot, capacity and kind of power equipment, outline sketch of machinery equipment, and such other general information as may be necessary

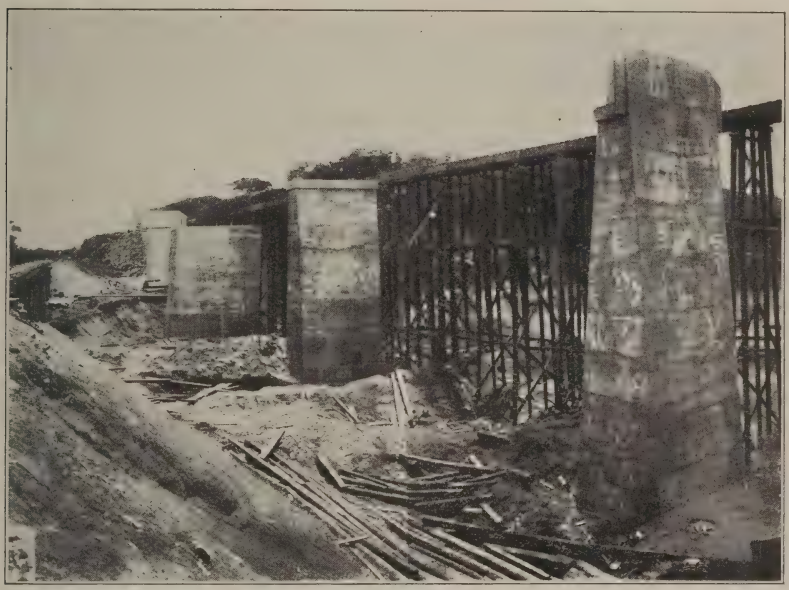


Fig. 3. Reinforced Concrete Piers for Bascule Bridge over Gatun River, March, 1911.

to enable the design and estimates of quantities to be checked by the engineer.

(*B*) A design in sufficient detail to illustrate the construction, the method of operation, and main mechanism.

(*C*) Detailed estimate of quantities and costs. The successful bidder to promptly furnish a complete set of tracings or Vandyke negatives of bridge, and three sets of blue prints.

DESIGN.

11. (*A*) Structural steel, pins, and cast steel for structural purposes. (Trunnions, bearings, etc.)

Case 1.—Bridge closed and subject to dead and live loads; loads and unit stresses as for stationary structures in accordance with

Cooper's E-50 loading, and general specifications for steel railroad bridges of the American Railway Engineering and Maintenance of Way Association.

Case 2, bridge-moving.—Find the actual maximum stress in each member that will occur at any angle of opening during the movement of the bridge, and apply allowable unit stresses equal to five-sixths of the dead-load unit stresses for stationary structures. Neglect reversal of stress, as this takes place slowly.

Wind load.—Fifteen pounds per square foot of entire surface of moving leaf.

Journal bearing on trunnions under motion—One thousand seven hundred pounds per square inch average, 2,000 pounds per square inch maximum.

Rollers with one contact surface—For rolling bascule bridges rollers may be of cast steel with solid or open webs, or may be built up with solid webs and hardened-steel rims. They shall be designed to safely withstand all stresses which may be developed in them by the loads specified above. Track plates on supporting or segmental girders of rolling bascule bridges may be of structural, cast, hardened, or other special steel. They shall have a depth sufficient to properly distribute the pressure to the girder to which they are connected, which depth may vary generally between the limits of one-fourth and three-fourths the width of moving bearing contact. The width shall be not less than given by the following formula:

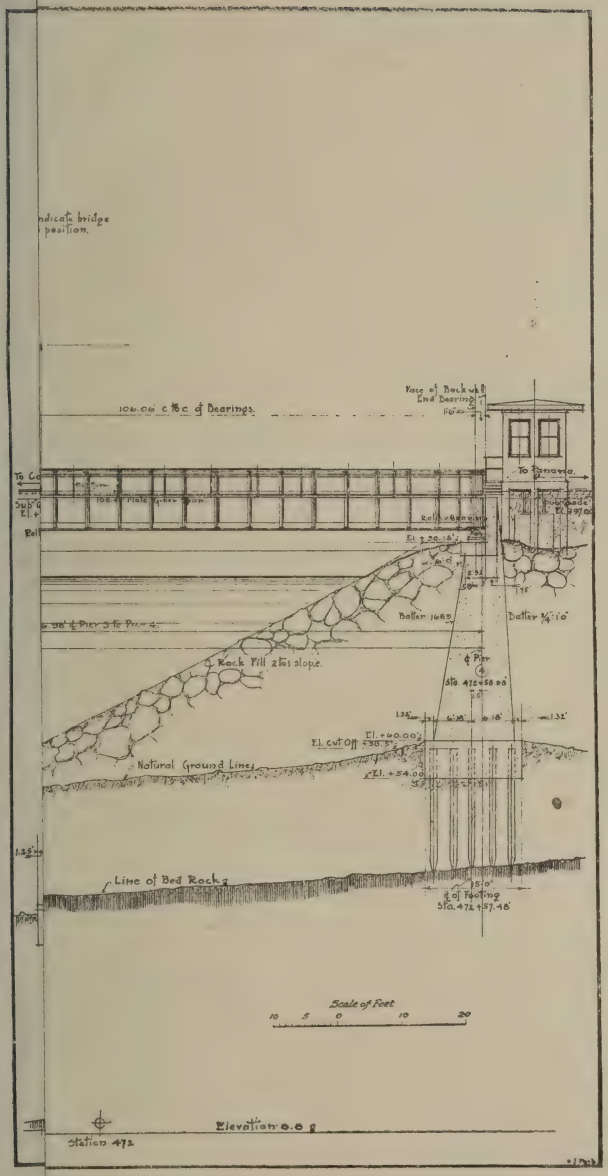
$$f = \sqrt{\frac{P^2}{R} \times \frac{9E}{64\ S^3\ d}}$$

where f equals width of face of track plate in rolling contact,
d equals depth of track plate in inches,
P equals total load in pounds on track plate,
R equals radius of segmental girder or roller in inches,
E equals modulus of elasticity assumed at 30,000,000,
S equals one-half of the ultimate strength of material used.
All dimensions are in inches.

The bearing pressure of the track plates on the webs and flanges of the girder (neglecting outstanding leg of angles) shall not exceed the permissible unit stresses in compression in the flanges of the girders. This pressure shall be assumed as distributed over a length in inches not greater than that determined from the formula:

$$L = 2\sqrt[3]{\frac{3RP}{E} \times \frac{d}{f}} + 2d.$$

These track plates must be so connected to the girders as to prevent any possibility of working loose or being damaged when subjected to all stresses of whatever character resulting from the rolling contact and weight.



ilroad. General drawing

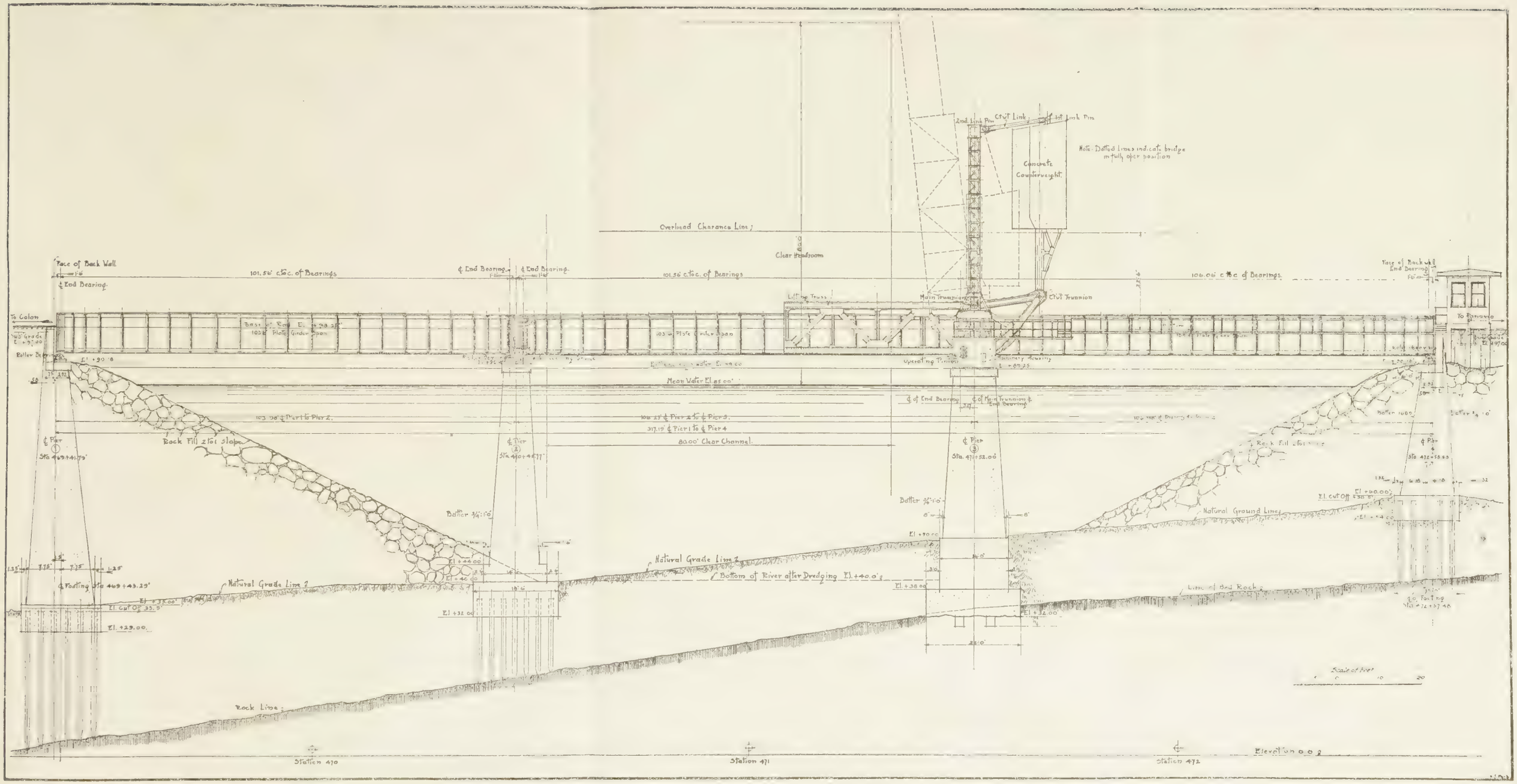


Fig. 4. Bascule Bridge over Gatun River, near Monte Lirio station. Constructed in connection with the relocation of the Panama Railroad. General drawing showing bridge spans, river profile and water surface elevations.

(*B*) *Operating machinery.*—Figure the strength of the operating machinery for a wind load of 15 pounds per square foot, as above, using the unit stresses given in specifications for movable bridges by C. C. Schneider.

(*C*) *Motors.*—Figure motors strong enough to operate the bridge against a wind pressure of 10 pounds per square foot of moving leaf in one and one-half minutes. For general description of motors desired and character of tests required, see I. C. C. Circular No. 627, pp. 16 and 17.

MISCELLANEOUS EQUIPMENT.

12. All necessary electrical equipment, operator's house, machinery inclosures, stairways, ladders, railings, indicators and signals, and all other materials necessary to erect the bridge complete and ready for traffic in accordance with the specifications, shall be furnished.

The bidder should furnish with bids a list, with prices, of all special tools which are necessary to be used in erecting the bridge, the Commission reserving the right to purchase these tools, at the prices stated, in its option. The price stated for the tools to include delivery free of all charges at Colon or Port of Ancon.

POWER.

13. There will be available as a source of power for the operation of this bridge alternating electric current, furnished from the transmission line which will pass along the right of way of the Panama Railroad between Gatun and Miraflores. The contractor shall furnish all necessary electric motors and other electrical equipment and controlling apparatus for the safe and satisfactory operation of this bridge. All electric motors and other electrical equipment in connection with the operation of the bridge shall operate from 220 volt, 25 cycle, 3-phase, alternating current. All necessary transformers will be furnished and installed by the Commission.

CLEARANCE.

14. Lifting apparatus to be designed to provide 22 feet overhead clearance, above top of rail, and 7½ feet clearance either side of center of track.

INTERLOCKING AND SIGNALS.

15. The operating controller of interlocking mechanism, and the railroad signals protecting the bridge, should be so interlocked that it will be impossible to draw the lock until the railroad signals have gone to "Danger," or to start the motor before the lock has actually been thrown.

It should be impossible to move the interlocking until the bridge has been completely closed, or to set the railroad signals at "Proceed" before the lock has been thrown.

As a complete system of automatic signals has been ordered to cover this part of the relocated line, it will therefore be unnecessary for the contractor to supply the usual distant "Caution" and

"Danger" signals, other than it should be so designed that it can be connected with the ordinary mechanical interlocking plant.

SPEED OF OPERATION.

16. Sufficient power should be installed which will permit the span to be raised from the fully closed position in from one to two minutes, and in like manner closed from the fully open position in same time.

PAINTING.

17. All metal work to be given one good coat of red lead before leaving the shop.

Fig. 5. Bascule Bridge over Gatun River, Panama Railroad. View of Completed Structure.

CLIMATIC CONDITIONS.

18. The wiring and entire apparatus will be subjected to more severe climatic conditions than usually encountered in the United States.

The temperature will fluctuate between approximately 75° and 125° F. There are heavy, warm, tropical rains, severe electrical disturbances, and the air is saturated with moisture.

These conditions must be taken into consideration in designing all equipment.

ACCEPTANCE.

20. Bascule to be erected by the Commission at the Gatun River in operative position and operated under required conditions for a

period of ten days' satisfactory operation, and all defects developing to be corrected at contractor's expense, before final acceptance.

MATERIAL AND WORKMANSHIP.

21. The material and workmanship utilized in the construction of the bascule to be in accordance with best modern practice, first class in every particular, and subject to inspection at the works by competent authorized inspector, and any material or workmanship contrary to the above intent to be subject to correction.

The different classes of material shall conform to the following specifications for structural steel:

Fig. 6. Bascule Bridge over Gatun River, Panama Railroad. End View of Completed Structure.

Rivet steel and steel castings, standard specifications for structural steel for bridges of the American Society for Testing Materials.

For steel forgings, I. C. C. standard specifications No. 261.

For cast iron, I. C. C. standard specifications No. 212.

For axle steel, I. C. C. standard specifications No. 262.

For workmanship of structural work (general reaming of rivet holes is required), I. C. C. standard specifications No. 254.

For brass, I. C. C. standard specifications No. 97.

Attention is called to clause 8, in which it is specified that preference will be given to designs in which the operating machinery is located on the stationary part of the structure. This

feature is standard in the type of Strauss design used for this bridge and has, in fact, become standard in all the Strauss bascules. The advantages of thus locating the operating machinery are as follows: Convenient inspection can be made at all times. During repairs and renewals the bascule can remain open and the operating machinery remains in its normal position, thereby facilitating such work. The electric connections are more secure and in case of armature renewals the stationary position of the motor is particularly advantageous, since the bridge can be operated by hand-power during this period without interference with such renewals. The oil cups retain their contents.

Attention is also called to clause 18, wherein the climatic conditions are referred to, and in the design of the structure these conditions were carefully borne in mind, in such details as insulation, motor windings, absence of springs, adequate housing of the operating machinery, etc., while in the most important elements of movement the design selected naturally lends itself to the fulfilment of these conditions, since the structure turns on trunnions which are absolutely protected against the influence of moisture and since they are housed in compact, well lubricated bearings, provided with phosphor bronze bushings.

The structure was completed in August, 1913, but owing to the absence of electric power was operated by hand for several weeks. The complete opening or closing of the leaf requires twenty minutes by hand power and about one and one-half minutes by electric power.

The total weight of the structural steel of the bascule including main girders, lifting trusses, trunnion posts, counterweight frame towers, etc., is approximately 313,000 pounds; the trunnions and bearings and sleeves for same weigh approximately 15,000 pounds; the operating machinery, including front end locks, weighs approximately 48,000 pounds. The concrete in the counterweight weighs approximately 925,000 pounds.

The entire work was under the direct charge of Lieut. Frederick Mears, formerly the Chief Engineer of the Panama Railroad, who is now one of the members of the Engineering Board of Three of the Government Railway in Alaska. The bascule was designed by the Strauss Bascule Bridge Co. of Chicago, and the steel fabricated by the Penn Bridge Co. of Beaver Falls and erected by the Panama R. R. Co.

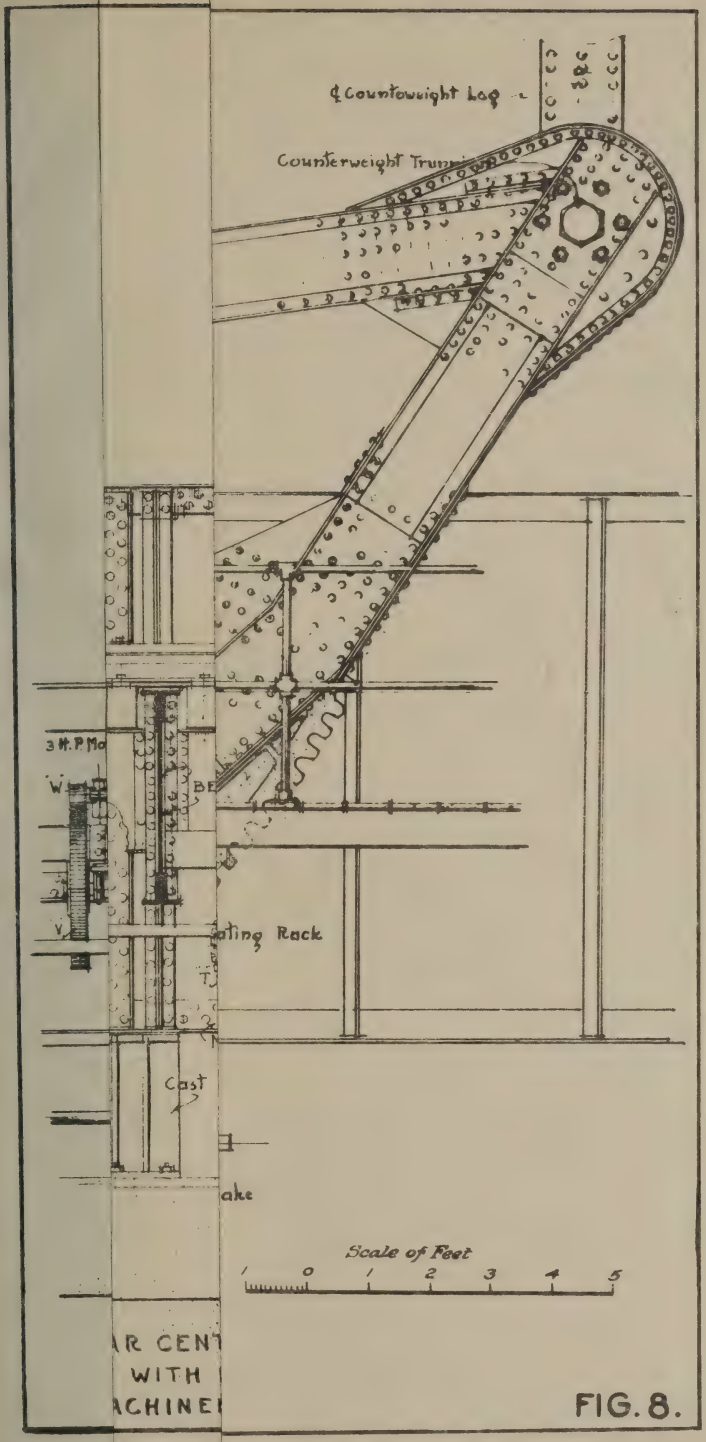
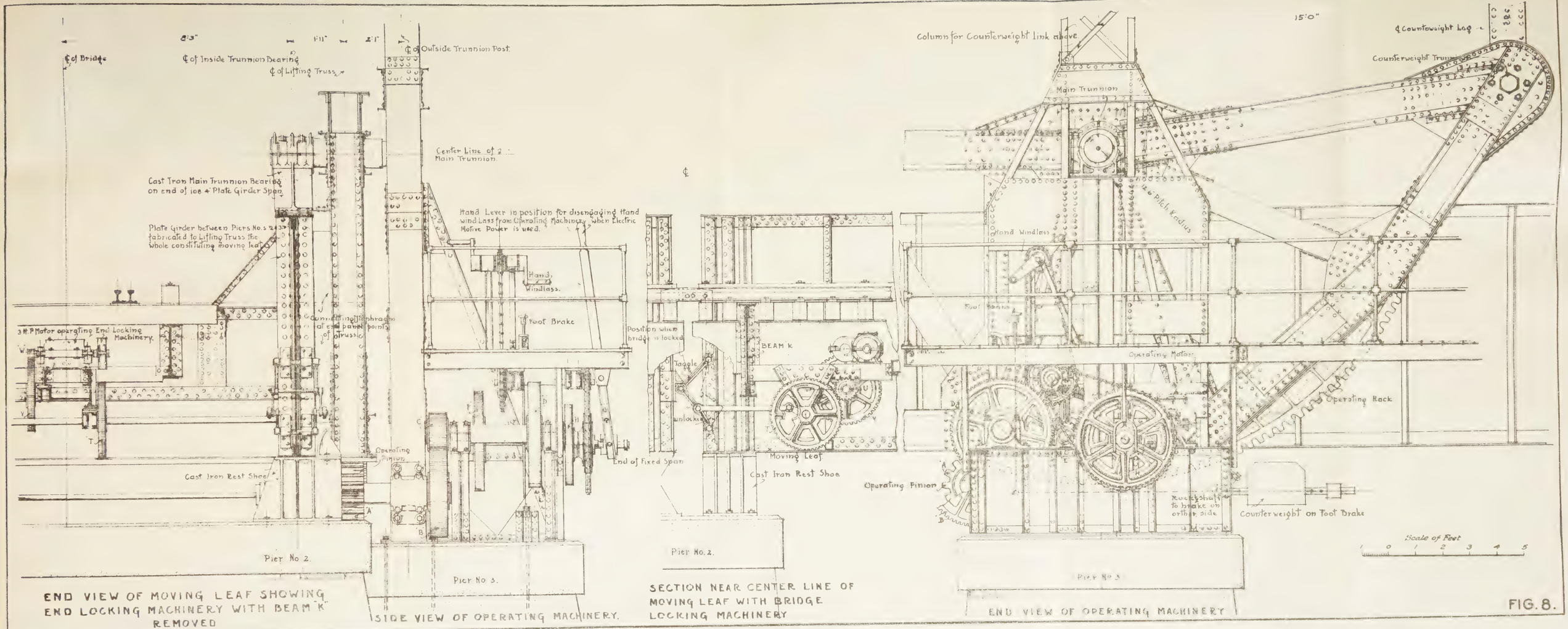

℄ Counterweight Leg
Counterweight Trun
3 H.P Mo
W
BE
ating Rack
Cast
ake
Scale of Feet
/ 0 / 2 3 4 5

FIG. 8.


℄ of Bridge
℄ of Inside Trunnion Bearing
℄ of Lifting Truss
℄ of Outside Trunnion Post
Center Line of 2 Main Trunnion.
Cast Iron Main Trunnion Bearing on end of 108 4' Plate Girder Span
Plate Girder between Piers No.s 2 & 3 fabricated to Lifting Truss the whole constituting moving leaf
3 H.P Motor operating End Locking Machinery.
Connecting Diaphragm at end panel points of Truss
Operating Pinion
Cast Iron Rest Shoe
Pier No. 2.
Pier No. 3.
Hand Lever in position for disengaging Hand wind Lass from Operating Machinery when Electric Motive Power is used.
Hand Windlass.
Foot Brake
Position when bridge is locked
Toggle
Unlocked
BEAM K
End of Fixed Span
Moving Leaf
Cast Iron Rest Shoe
Pier No. 2.
Column for Counterweight Link above
Main Trunnion
Hand Windlass
Foot Brake
Operating Motor
Operating Pinion K
Rock Shaft to brake on other side
Counterweight on Foot Brake
Operating Rack
℄ Counterweight Lug
Counterweight Truss
Pier No. 3
Scale of Feet
0 1 2 3 4 5

END VIEW OF MOVING LEAF SHOWING END LOCKING MACHINERY WITH BEAM "K" REMOVED

SIDE VIEW OF OPERATING MACHINERY.

SECTION NEAR CENTER LINE OF MOVING LEAF WITH BRIDGE LOCKING MACHINERY

END VIEW OF OPERATING MACHINERY

FIG. 8.

The Rise of the Bed of the Yellow River by the Deposit of Sediment.*

BY

Brig. Gen. WILLIAM L. SIBERT,
U. S. Army.

The Board of Engineers who made the report on the Huai River Conservancy Project believe there is nothing in the facts set forth in that report which can reasonably be construed as condemnatory of levee construction on the Mississippi or on other rivers. The members of the Board greatly regret that time was not available for them to personally examine, in greater detail, the physical conditions on the Yellow River, and to collect and publish all the available physical data connected with this most unusual stream.

While the problems of the Yellow River were discussed at some length in the report, the Board was unable to devote the time or space necessary for a complete discussion of even the limited data which it had secured. Although these problems are of great technical interest they are not of corresponding importance to the main problem before the Board for solution.

The criticism of the report is based primarily on the assumption that the flood discharge of the Yellow River is "over 1,000,000 second feet." The descriptions by Mr. Elias of the flow of the river after its last change of channel and before it was dyked constitute quite uncertain premises for such a definite assumption.

T. W. Kingsmill measured the flow of the Yellow River at Tsinan in April, 1887, and found a discharge of 23,000 second feet. Capt. W. F. Tyler made two measurements on March 28,

*Discussion of the article entitled "The Effect of Levees on the Flow of Sediment in Rivers. A discussion of the Article on 'The Huai River Conservancy Project,'" printed in PROFESSIONAL MEMOIRS No. 32. Contributed by General Sibert for the Red Cross Board of Engineers.

1903, at Li-Ching, and on April 20, 1903, at Lo-K'on, and found 20,400 and 30,400 second feet, respectively.

The hydrograph of the Yangtse River (see Plate XV—Huai River Report) shows that the floods on that river occur in July, August and September, and that the extreme low water period is early in February. The flow in April is about a mean for the year. The flow of the Yellow River is believed to be similar in character, hence the flow of 30,000 second feet or less is probably the mean flow for the lower Yellow River. No records were found as to its flood discharge.

The Yellow River drains the desert country of Mongolia and Tibet. The rainfall near the outlet from the mountains is under 20 inches per annum, and the precipitation, even in the mountains, occurs principally in the warm weather, and is believed to be very low. No tributaries enter the Yellow River after it leaves the mountains.

It seems doubtful whether the most devastating floods have ever exceeded a discharge of 300,000 or 400,000 second feet at the mountain outlet.

If the section of the Yellow River bed at Antung, about 60 miles above Mengkung, be considered (see Plate III, Professional Memoirs, No. 31), it will be noted that the section of the channel to the top of the dikes is also about 8,000 square feet, and the impossible velocity of 125 feet per second would be required to pass the estimated discharge of 1,000,000 second feet through the section. If the section were excavated to the country level for the entire distance between the dikes, the area would be only 16,000 square feet, and the velocity necessary to pass assumed flood discharge would be 67½ feet per second. To reduce this velocity to 10 feet per second, the bed would have to scour to a depth of about 146 feet, or more than 100 feet below sea-level, between the dikes which are here only 575 feet apart. This narrow section extends for 20 miles or more with some considerable bends in the dike lines, and that such a river section in the soft deposits of the Chinese plain could ever have been maintained between dikes with such a discharge and consequent velocity, is inconceivable.

The answer is that the flood discharge in the lower course of the Yellow River, east of Huntze Lake, was probably seldom one-tenth of the amount estimated by Colonel Townsend. When the river exceeded this it probably broke the dikes and flowed into Huntze Lake.

Soundings taken in the bends of the present channel of the Yellow River near Kaifengfu showed a depth of from 12 to 14 feet with the river 4 or 5 feet above low water. The Board made a trip in a boat over a 35-mile stretch of river in this vicinity. The variation between high and low water was stated to be only 6 or 7 feet in this reach of river.

An old Chinese map published in the report will show that the Yellow River in extreme floods found an outlet through Huntze Lake, and from that, by a series of lakes into the Yangtse River; by which route during floods, undoubtedly the larger part of the flood waters of the Yellow River travelled. The Chinese first tried to bar the passage of this water from Huntze Lake into the lakes to the south by the construction of a cut-stone wall along the east bank of Huntze Lake. They afterward made openings through this wall and provided for the passage of water to the Yangtse. In these openings were constructed removable dams that were taken out during floods and replaced when the river reached a medium stage. There were also removable dams built across the connection between the Yellow River and Huntze Lake, for the purpose of barring the flow of the Yellow River into that lake during floods, which dams were taken out during low water for navigation. History recites that all these dams often broke. The Board found the bed of Huntze Lake from 15 to 20 feet higher than the country to the east of the Grand Canal. They found the bed of Paoying Lake and Kaoyu Lake from 6 to 7 feet above the country to the east of the Grand Canal. This divided outlet of the Yellow River described above could easily have caused the silting of the Yellow River channel from the Grand Canal to the Sea during floods, but the concentration of the entire flow at low and medium stages through the Yellow River channel to the sea would probably have removed such deposits.

The conclusions in this case were not based entirely upon the sections of the Yellow River that were printed. The Board obtained from the Chief Engineer of the Tsiensin Pukow Railway the profile of the old bed of the Yellow River where that road crossed it. It also obtained from the Chief Engineer of the railway extending from Suchow to Kaifeng a profile of two or three crossings of the old Yellow River bed between Suchow and the Grand Canal. The Board also visited and had sections made of the old Yellow River bed near the point of its last diversion. In

addition to this there was available the evidence recorded in the reports and papers of Messrs. DeRijke, T. W. Kingsmill and W. F. Tyler, engineers, who had personally examined the lower river. From all of this data it was concluded that the Yellow River had elevated its bed above the surrounding country. It was appreciated that this was an unusual condition, and that it was due to the fact that there was not sufficient slope in the plain crossed by the Yellow River to enable that stream to carry its load of silt to the sea. The reason for this was sought. Such evidence of geologists as could be obtained was considered, and the conclusion was reached that this abnormal condition of the Yellow River was either due to a material addition to its length in a relatively short space of time, or to a depression of the plain that it crossed in some adjustment of the earth's crust. The preponderance of evidence available seemed to favor the former assumption. Whatever be the cause, it was evident that the Yellow River was not flowing through a delta plain built by itself that remained at the same elevation as when built. If a stream has a practically constant length and the middle course of its bed is depressed by geological subsidence, that stream would naturally build up its bed and flood plain until it created the necessary gradient. If its length was suddenly increased by 300 or 400 miles, the same thing would happen. It seems now, from further geological research, that the evidence is in favor of the assumption that the alluvial plain has subsided, as the mountain region of Shansi has risen, and is still subsiding, and that the district of the Huai River and the Huntze Lake lies in the extreme southeastern part of a down-warped or depressed area. This depression was evidently fast enough to prevent the Yellow River from maintaining a normal delta slope, thus continuing an abnormal condition. It matters not whether the proper gradient does not exist because the alluvial plain is lowered faster than the river can build it up, or because of the fact that the river received a sudden and material increase of length. The fact that the Yellow River has raised its bed above the surrounding country seemed to the Board to be established by the data that it had before it.

That the abandoned bed of a stream is frequently partially filled is a matter of common observation. The apparent fill of the bed of the Colorado River shown in Colonel Townsend's illustrations Nos. 1, 2 and 3 is due principally to the erosion of

the new channel, under the excessive grade into the Salton Sea and but slightly to an actual filling of the bed itself.

Some filling near the point of departure from the old course of the Yellow River is probable, although the personal examination of the Board shows that this occurred only in the old channel, and to a very limited extent and did not and could not have extended up to the flood plains of the old Yellow River bed.

When the Mississippi River abandons an old channel, it fills the old channel at points adjacent to the points of departure, but evidence of its former occupancy is seen in bayous and lakes which occupy its former course. No such conditions are found in the old bed occupied from 1324 to 1853. Kingsmill writes of an ancient elevated bed of the Yellow River near Pohsing, and Tyler describes three of such raised beds visited by him, one probably occupied by the Yellow River 600 B. C. Another occupied by the Yellow River about 1194, along which a well defined dike could still be traced. Bearing in mind the fact that the Yellow River had a free outlet into the open country in 1853, is it possible that this stream, in abandoning its course of 1324 to 1853 would so far depart from the normal action of rivers that it would fill its old bed for 350 miles to an elevation of from 0 to 25 feet above the adjacent country level and create at the same time a fairly uniform and well defined channel for the whole distance, with a system of natural dikes and flood plains?

These features of the old river bed are explainable only on the assumption that they were built by the river flowing normally between restraining dikes.

The breaches of the dikes which have resulted in the burial of native villages to the eaves of the houses and of bridges to the springs of their arches, indicate fills of silt above the country level of from 1 to 10 feet. This can be reasonably accounted for only by the hypothesis that the bed of the river, composed of friable loess grains, is above the surrounding country, and when a breach occurs is washed in the form of semi-liquid mud over the adjacent lower lands. The billion cubic yards of mud discharged by the Yellow River in 1898 from the breach below Chi-nan can not reasonably be accounted for on any other rational hypothesis. The deposits are too great and too widely extended to be the result of the normal deposit of material suspended in the flood waters. The reduction of the bed of the

river and the consequent reduction in the flood heights above the point of diversion in 1853 is due to the same cause.

It should also be noted that "rafts" are not a feature encountered in the rivers of China, where fuel is so scarce and labor is so cheap that not a sign of drift is allowed to float in the rivers.

The Board has not assumed that the various changes in the course of the Yellow River across the Great Plain of China, within historic times was due to "Upheavals of the earth's crust," as is perfectly obvious from a careful reading of the report. The Board has advanced such a reason for the radical change in the course of the Yellow River which apparently did occur in the mountains of northwestern Shansi in prehistoric times.

It may be noted that a similar development of the strata apparently changed the course of the prehistoric river that drained the southern portion of the Appalachians in our own country and caused its waters to change from the southwest and an outlet in the Gulf and Lower Mississippi River to the northwest and an outlet in the Ohio River, giving rise to the Tennessee River (see "Geology," Chamberlain and Salisbury, Vol. I, pages 164-168).

It is regretted that the sections taken of the Yellow River near Kaifengfu can not be found. The Board will write to China for copies and have them printed.

Miscellaneous Notes.

The Editors are pleased to be able to present in this issue to the readers of PROFESSIONAL MEMOIRS an article from the pen of Dr. Elmer L. Corthell, which they know will be read with pleasure and profit by all. This paper was originally intended for publication by the International Association of Navigation Congresses as an ad interem paper, but such action was made impossible by reason of the European War; an opportunity was thus afforded PROFESSIONAL MEMOIRS, through Doctor Corthell's courtesy, to give to its readers at first hand the benefits that were at his disposal.

It should be added that this paper will not, although Doctor Corthell is a subscriber to PROFESSIONAL MEMOIRS, be considered in making awards for prizes announced in No. 30.

Errata, No. 33.

Page 312, line 29, should read: 36,816 feet B. M. Framed Timber, heart, $2,577.12, and the total, $2,740,060.74, instead of $2,740,-200.74.

Page 332, line 4: 17,500 should read 15,500.

Book Reviews.

THE PANAMA CANAL.—Comprising Its History and Construction and Its Relation to the Navy, International Law and Commerce. By Reuben E. Bakenhus, S. B., Corps of Civil Engineers, United States Navy; Harry S. Knapp, Captain, United States Navy, and Emory R. Johnson, Ph. D., Sc. D., Special Commissioner on Panama Canal Traffic and Tolls, 1911-1913. 240 pages of text and an appendix of six pages; maps and illustrations. Pages 6 by 9. Cloth binding. John Wiley and Sons, New York. Price, $2.50, postpaid.

This book is in its first edition, first thousand. It is printed in clear large type and presents a pleasing appearance. The illustrations are twenty in number and add to the interest of the book. There are three plates of maps and three plates of plans and sections of the canal and locks.

In their introduction, the authors of this work state as their purpose the exclusion of all non-essentials, however interesting, and the limiting of themselves to the statement of principles, laws and facts to which the canal owes its existence. The result is a very illuminating and interesting volume in six parts, to which is appended a number of entirely reliable maps and plans by which the reader is able to understand clearly the subject matter of the work.

Part One embraces a clear, brief but sufficient history of the various projects for constructing an Isthmian Canal from the earliest times down to the transfer of all control to the United States. Part Two discusses all essential matters relating to the choice of route, the design and construction of the canal and the reasons that governed the authorities in all their final decisions. Part Three deals with the important problem of sanitation, the cost, the equipment and the various elements of success entering into the completion of the work.

The author of these three chapters, Mr. Bakenhus, has shown unusual skill in selecting and condensing the great mass of information here contained and at the same time presenting it in such an interesting form for the average reader. One notes, however, the slight attention given by the author to the great work of electricity in the construction and operation of the canal.

In his chapter on "The Navy and the Panama Canal," Captain

Knapp, after pointing out the many advantages accruing to the Navy by reason of the completed canal, convincingly shows that although it may effect some changes in the disposition of the fleet in time of peace, yet the composition of the fleet as well as the number and proportion of vessels will be changed very little. The reason given for this is, that the combined fleet at present does not equal the "Standard Atlantic Fleet" alone, and that the Navy in order to be effective must be sufficiently large and varied to meet the possibilities of simultaneous attack on both oceans.

The chapter on "Panama and International Law," by the same author, is a forceful justification of the attitude taken by the United States in the matter of the various treaty relations pertaining to transcontinental canals. It contains the text of these treaties together with many pointed comments thereon.

The next edition of this work will undoubtedly contain a revision of the final chapter by Professor Johnson on the commercial importance of the canal. Much data not available when the present work was written will make this chapter highly interesting in the future edition.

If this book were the only source of information extant regarding the canal the first question that would be asked after reading it would be: Who built the canal? He looks in vain who would find the name of a single individual directly connected with its construction since the American occupation.—H. A. B.

THE THEORY AND DESIGN OF STRUCTURES.—A Textbook, for the use of Students, Draughtsmen and Engineers engaged in Constructional Work. By Ewart S. Andrews, B. Sc. Eng. (Lond.), Late Demonstrator and Lecturer in the Engineering Department of University College, London; Lecturer in Theory and Design of Structures at the Goldsmiths College, New Cross. Third Edition. Cloth 6 by 9 inches, xii+618 pages, 289 illustrations. D. Van Nostrand Company, New York. Price, $3.75 net.

The third edition of Andrews "The Theory and Design of Structures" consists of 18 chapters of text revised from the earlier editions of the book, a table of British standard sections, about one hundred and thirty problems, exclusive of the exercises that are worked out in full, and an appendix.

The appendix treats the following subjects in elaboration of the corresponding discussions in the body of the work: (1) Repetition or Variation of Stresses. This comprises a very brief statement of recent experiments that have been made to determine the effect of the repetition of stresses upon the strengths of materials. (2) Wind Pressures—Stanton's Experiments. This abstracts Stanton's experiments to determine the suction effect of

the wind on the leeward side of structures that are exposed to its action. It discusses the changes in the intensities of the stresses in the members of the trusses that are thus occasioned. (3) Stresses in Curved Beams, A Template for Bending Moment Diagrams, The Deflection of Beams, and Framed Structures, are the headings of notes in explanation of short cuts to the graphical determination of stresses.

The book is well suited for use as a text-book. It develops the ordinary theories of structural design principally for the graphical solution of engineering problems. However, their analytical solution also, is explained. In elaboration of the theories many exercises are worked out, so that the student becomes familiar with their various uses, and is enabled to undertake the complete design of an ordinary framed structure, in accordance with British practice.

The book as a whole is excellent; although it does not attempt to exhaust its subject, it does develop it sufficiently for its purpose, which for the student, is to furnish sound theory, and for the designer, to exhibit the elements of accepted British practice.

It is profusely illustrated, but neither the arrangement nor the quality of the figures is in keeping with the excellence of the text. The drawings seldom occur on the pages where they are most needed. The reader must waste time in studying their description by turning the pages to find the figures. Duplicated drawings would enhance the usefulness of the book. The lettering is poor, and thus there is presented to the student a low ideal of the draughtsman's skill.

The text comprises eighteen chapters, as follows:

Chap. 1, Stress, Strain and Elasticity; Chap. 2, Principles of Design, Working Stresses, Wind Pressures; Chap. 3, Forces. Areas and Moments; Chap. 4, Riveted Joints and Connections; Chap. 5, Bending Moments and Shearing Forces in Beams; Chap. 6, Stresses in Beams; Chap. 7, Bending Moments and Shearing Forces in Beams; Chap. 8, Deflections of Beams; Chap. 9, Fixed and Continuous Beams; Chap. 10, Distribution of Shearing Stresses in Beams; Chap. 11, Framed Structures; Chap. 12, Columns, Stanchions and Struts; Chap. 13, Suspension Bridges and Arches; Chap. 14, Masonry Structures; Chap. 15, Reinforced Concrete and Similar Structures; Chap. 16, Design of Steelwork for Buildings; Chap. 17, Design of Roofs; Chap. 18, Design of Bridges and Girders.

The treatment of the subjects of the above chapters is uniformly admirable.

Three chapters stand out prominently, Chapters 2, 15 and 17. The discussion of reinforced concrete, though full and complete as to theory and design, omits a description of the ingredients of concrete and of the principles and methods of mixing, of

which the average student is woefully ignorant. The chapters on working stresses and wind pressures and the design of roofs are especially to be commended. The explanations therein of the necessity for making allowances for live loads and for the variation of stresses that reduce the factors of safety as commonly figured, and of the various methods of dimensioning truss members accordingly, are fully set forth and emphasized as a warning against hasty approximations.

Typographical errors are noted as follows: p. 37, line 17, *place* is omitted; p. 61, figure *d* should be *a*; p. 144, line 12, *longitudinal cross* should be interpolated before *section;* p. 144, line 14, same as before; p. 145, figure appears not to be a cantilever. —F. B. D.

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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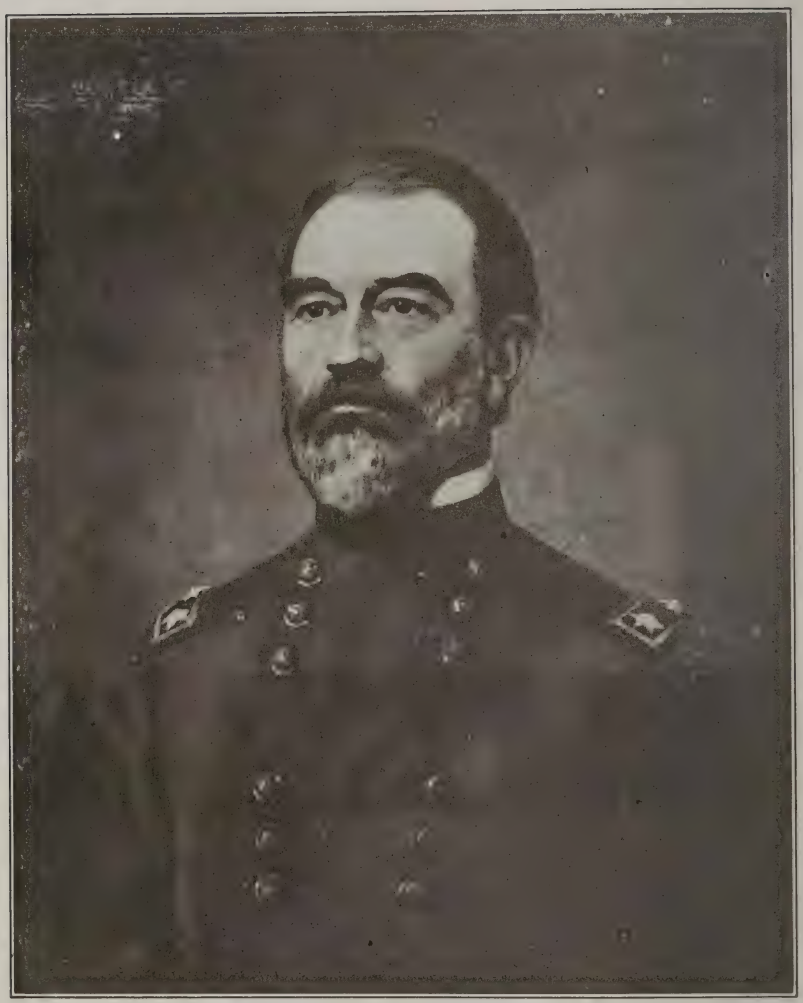


ZEALOUS BATES TOWER

CORPS OF ENGINEERS

BORN 1819—DIED 1900

Cadet, U. S. M. A., 1837-1841.
Second Lieutenant, Corps of Engineers, 1841.
Retired as Colonel, Corps of Engineers, 1883.
Brigadier General, U. S. Volunteers, 1861.
Brevet Major General, U. S. Army, 1865.
Brevet Major General, U. S. Volunteers, 1865.

SEE PAGE 512

Vertically Framed Mitering Lock Gates.

BY

Mr. MALCOLM ELLIOTT,
Junior Engineer; Assoc. M. Am. Soc. C. E.

All mitering lock gates may be broadly included in one or the other of two types: *horizontally framed* and *vertically framed* (so called). The purpose of this article is to point out some of the advantages of the vertical frame type for certain conditions. A brief description of the two types and of the manner in which each carries the loads to which it is subjected is first given.

A horizontally framed gate is one which resists the water pressure by means of a series of *horizontal* members more or less uniformly distributed from the bottom of the gate to the top. Each horizontal member receives the load on its own portion of the gate and is supported at one end by a corresponding member of the opposite gate leaf and at the other end by the lock wall.

The so-called "vertically framed" gate resists the water pressure in an entirely different manner. In this type the loads are carried by a series of *vertical* girders more or less uniformly distributed throughout the length of the gate. These vertical girders are supported at the bottom by a sill anchored into the floor of the lock, and at the top by a horizontal member which is in turn supported in the same way as are the horizontal members of a horizontally framed gate.

Fig. 1 shows a gate in its mitered position and may represent either the top horizontal member of a vertically framed gate or any horizontal member of a horizontally framed gate. The loads on the gate cause a combined direct thrust and bending in the horizontal members. It will be found that in all cases the stress in the upstream flange increases from either end of the gate to a maximum at the center and that the compression in the downstream flange decreases from the ends and may become negative—that is to say, a tensile stress at the center. In practice the flange angles at least

and, usually, one cover plate extend the entire length of a girder. It will usually be found, therefore, that there is some part of the downstream flange where the metal present is much greater than required for the stresses at that part. A part of the downstream flange will be superfluous, therefore, as far as the flange stresses alone are concerned.

This apparent waste of metal can be avoided by building the horizontal member in the shape of a circular arch. Fig. 2 shows a gate of this shape. If the points of contact at the miter and quoin ends are located on the center line of the arch it easily may be proved that the thrust is evenly distributed throughout the cross-section of the arch at any point in the gate and is constant throughout the entire length of the gate. In fact, the arch thrust is the same in amount as would be the tension in a pipe of the same radius subjected to internal pressure. On account of the absence of combined bending and direct stresses, the arch type is more economical than the straight girder type as far as weight is concerned. There are, however, other things beside weight to be considered in choosing the best type of gate. Some of these will be mentioned later.

The foregoing remarks apply to any horizontal member of a lock gate, whether it be the single top member of a vertically framed gate or the horizontals of a horizontally framed gate. With the horizontally framed gate, each girder or arch carries a load depending upon its depth below the surface of the water and the width of sheathing it supports. With a vertically framed gate, the loads on the horizontal member are the reactions at the upper ends of the vertical girders. Fig. 3 shows the conditions of loading for this type of gate. The load on the top horizontal member, per unit of length, is equal to $\frac{62.5}{6h}(h_1^3 - h_2^3)$. When the upper pool is at the top of the gate and there is no lower pool acting, this expression becomes $62.5\ \frac{h^2}{6}$, which is one-third of the total water pressure on the gate. The other two-thirds of the pressure is carried to the sill.

The most important points to be considered in the choice of type for any particular gate are *cost* and *efficiency*, and of these it may be said, the cost of a gate is a relatively small part of the cost of the whole lock, whereas its efficiency is a very important factor in the lock's successful operation; and, therefore, no saving in cost on the gates is justified if it impairs their efficiency. On the other hand, it is just as true that if a saving can be made on the cost of

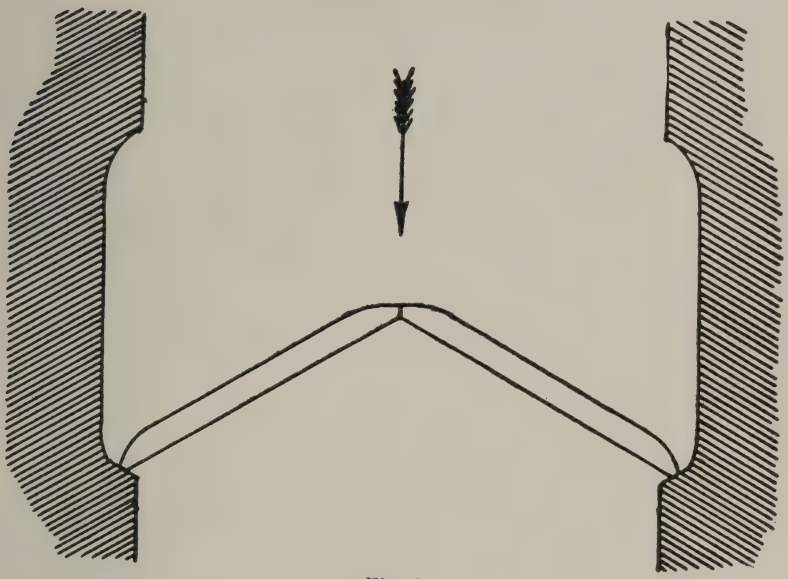


Fig. 1.

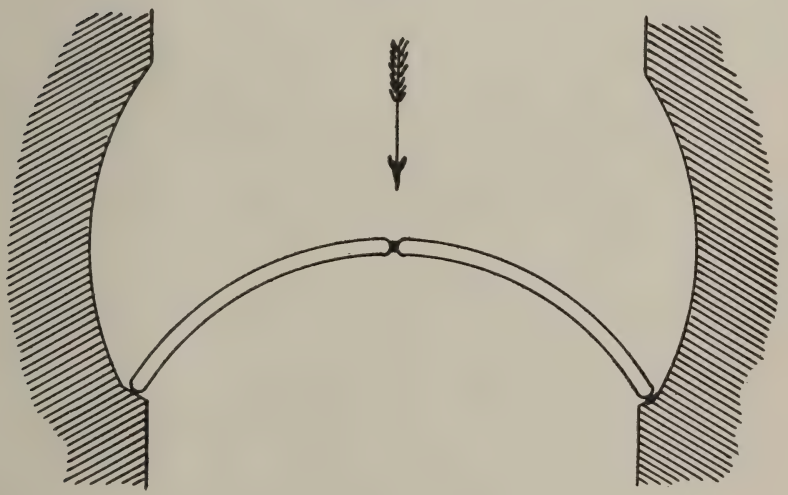


Fig. 2.

the lock gates without impairing the efficiency of the structure, the saving is justified even though it should amount to only a small percentage of the cost of the lock.

In comparing the relative costs of two types of lock gates the following elements should be considered: (1) the weight of the gate, (2) simplicity of design and ease of erection, and (3) the cost of that part of the walls and sills which is influenced by the design of the gate.

The advantage in weight for the two types for any particular lock can best be determined by making rough designs and comparing them. An inspection of the two types leads to the conclusion that for gates which are long in proportion to their height, vertical framing will weigh less, and that as the height is relatively increased the difference becomes less, until the two types will weigh alike and a further increase in height will result in an advantage in favor of horizontal framing. General Hodges* states that the weights will be equal when the height is two-thirds of the length. The writer has made comparisons between the two types for various dimensions and is of the opinion that a vertically framed gate will weigh the same as a horizontally framed gate, in which arches are used, when the height is about 75 per cent of the length and that when girders are used for the horizontal members the weights of the two types will be equal when the height is equal to the length.

Turning now to facility of construction—there can be no doubt that a vertically framed gate is much to be preferred. The conditions of loading and support are such that no extraordinary accuracy of workmanship is required. So far as water pressure is concerned, the gate is supported along the sill and at two points on the upper girder, while a horizontally framed gate is supported along three edges of the rectangular surface exposed to the water. It is clear that in order to obtain the support, in the latter case, extraordinary care in fabrication and erection is necessary. A vertically framed gate is well adapted to the use of adjustable diagonal bracing by means of which any sag or warp in the gate can be corrected. With large horizontally framed gates, it has not so far been practicable to use adjustable diagonals, and therefore the gate must be built so as to remain comparatively free from

*"Notes on Mitering Lock Gates," by First Lieut. Harry F. Hodges, Professional Papers of the Corps of Engineers, U. S. Army, No. 26; Washington, Government Printing Office, 1892.

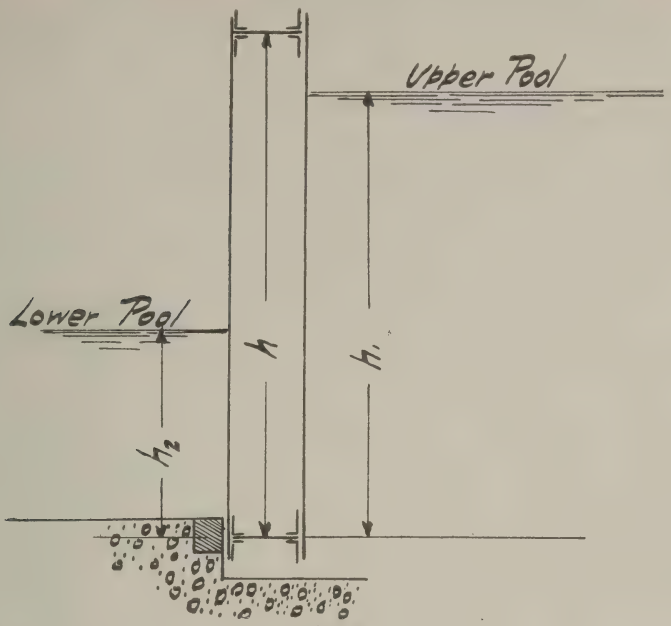


Fig. 3.

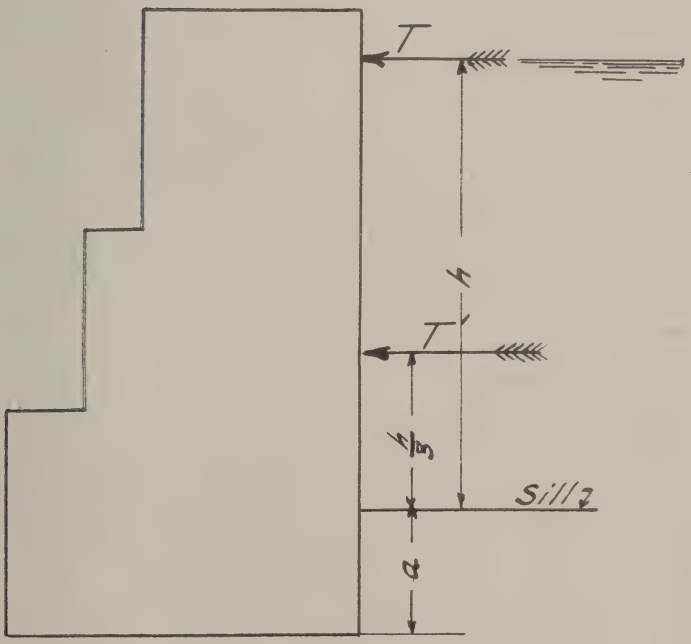


Fig. 4.

warp and sag, and this extra precision in workmanship and erection can not be obtained without extra cost.

Another cost-involving element in horizontally framed gates is the large amount of curved work usually found in this type. Even in those gates which have horizontal girders instead of arches, the ends are usually curved so as to reduce weight where the bending moment is small. The result is that the rivet holes in the sheathing plates are a little farther apart than those in the flange angles and the holes in the sheathing splices are still farther apart and will not agree with standard rivet spacing. This necessitates extra care in locating the rivet holes so that they will match up after assembling.

It is stated that for the Panama lock gates it was necessary to drill many holes from the solid after assembling the plates, in order to avoid excessive reaming which would have resulted in oval holes. The labor of bending the angles and plates and cutting the web plates on curves adds to the cost. None of this is necessary with vertical frames: first, because there is little curved work in the gate and, second, because, by being able to adjust the gate with diagonals, no extraordinary precision is required. The conclusion from the above is that where weight is the same the vertically framed gate is the cheaper on account of the facility of construction.

The type of gate selected affects the design of lock walls and sill. It is proper, therefore, to consider this effect in deciding upon type of gate for any given conditions. It relates principally to the amount of masonry and metal work required.

As to the volume of masonry in the walls, it has been stated, as a disadvantage of the vertically framed gates, that this type brings a heavy strain high up in the wall* due to one-third of the total thrust being concentrated at the top girder. While it is true that there is a heavy load on the upper part of the wall, there is, nevertheless, no greater overturning moment than would be the case with a horizontally framed gate.

Let Fig. 4 represent the wall at the quoin; T, the thrust from a vertically framed gate, and T′ the resultant of the thrust from a horizontally framed gate. If there is no water on the downstream side of the gate and if the water surface is level with the upper

*"The Improvement of Rivers," by Thomas and Watt, New York. John Wiley & Sons, 1903. Page 184.

girder, it is evident that T will be one-third of T′. The lever arm of T above the sill is three times that of T′; therefore, their overturning moments about the plane of the sill will be equal and there will be no choice between the two types as far as stability of the wall is concerned. About the base of the wall, which is always some distance, a, below the sill, a different result is obtained. With a vertically framed gate, the overturning moment about the base of the wall, if the sill foundation is self supporting, will be,

$$M=T\,(h+a)=Th+Ta.$$

For a horizontally framed gate,

$$M=T'\left(\frac{h}{3}+a\right)=Th+3Ta.$$

The second expression is evidently greater than the first and therefore, the advantage, if any, so far as the amount of masonry in the wall necessary to resist overturning is concerned, is with the vertically framed gate.

It has also been stated, with reference to the amount of masonry in the lock walls, that the top girder of a vertically framed gate is necessarily very deep on account of its heavy load and will require a deep recess in the wall, and therefore an added amount of masonry on the back of the wall in order not to decrease the top width and the available space for operating the gate. Even if a deeper recess were required, it is not clear why the masonry added to the back of the wall need be any greater in volume than that saved in the recess, but in reality the top girder of a vertically framed gate need not be any wider than the girders of a horizontal girder gate and usually encroaches on the wall much less than the arches of a horizontal arch gate. In the vertically framed upper gate recently designed for the new lock now being built at Louisville, Ky., the top girder is 6 feet wide. The length of the gate is about 62 feet. The depth of the girders should not in any case be less than about one-tenth of the span; therefore, in this case, the use of vertical framing does not encroach on the wall nor is there any reason why it should do so in any other case. Taking everything into consideration, it may be said that if there is any choice between the two types as to the volume of masonry in the walls, the choice is in favor of the vertical framing.

The volume of masonry required in the sill will probably be greater for a vertically framed gate, because for this type the stability of the gate is entirely dependent on the stability of the sill. The difference in the amount of sill masonry will, however, not be

as great as might at first sight appear. Since a horizontally framed gate is so designed as to take horizontal thrust, due to water pressure, throughout its entire height, each horizontal member taking its own share of the thrust, it is clear that the gate would be stable even if the sill were removed. Therefore, if the gate were perfectly fitted, a large amount of masonry would not be required in the sill. In practice, however, it may happen that the gate will come in contact with the sill before the two leaves are mitered throughout their entire height, in which case there will be a heavy pressure on the sill and sufficient masonry must be provided to resist it. For either type the masonry in the sill must be sufficient and so disposed as to stop effectively the flow of water from the upper to the lower pools and this consideration may result in as much masonry for one type as for the other.

Metal work is required in the walls and sills for both types of gates. For large horizontally framed gates there is required a metal hollow quoin in the walls, the metal being equal in length to the height of the gate. In vertically framed gates the thrust is concentrated at the top and is only one-third as great; therefore, the metal hollow quoin is very much smaller. On the other hand, more steel is required in the sills for a vertically framed gate on account of the heavier load. These items should about balance each other.

Taking all of the foregoing into consideration, the conclusions as to the relative cost of the two types are as follows:

1. The advantage in weight depends on the ratio of height to length. For a ratio less than 1 : 1, vertical framing will weigh less than horizontal girders. For a ratio less than 75 : 100 vertical girders will weigh less than horizontal arches.

2. In facility of construction the advantage is with the vertical framing.

3. In cost of walls and sills, there is no very marked difference between the two types.

The next point to be considered with reference to choice of type is the relative efficiency of the types available.

The word efficiency here is intended to mean the ability of the structure to perform its functions with the best results at the least expense for operation and maintenance. The service desired of a lock gate is for the two leaves to swing on their hinges like a door and, when closed, to meet each other accurately throughout their

W
6'0 to 10'0 Fl's
Upper Girder

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

PLAN

DOWNSTREAM ELEVATION

SECTION A-A

SECTION B-B

SECTION C-C

SECTION D-D

SECTION E-E
Scale-1½"=1'-0"

OHIO RIVER
LOCK AND DAM NO. 41.
UPPER GATE
GENERAL DRAWING

PREPARED UNDER DIRECTION OF THE
OHIO RIVER BOARD OF ENGINEERS ON LOCKS & DAMS

APPROVED BY THE OHIO RIVER BOARD, JANUARY 10, 1914.
J. B. Wills
Captain, Corps of Engineers,
Recorder.

APPROVED BY THE CHIEF OF
ENGINEERS, U. S. A., MARCH 13, 1914.

Fig. 5.

height at the center line of the lock at the same instant that the lower part of the gate comes in contact with the sill. Both horizontally and vertically framed gates can be depended on to perform this function with more or less success, but a vertically framed gate does so at a less cost for maintenance, for the following reasons:

1. Buoyancy chambers are usually necessary in large horizontally framed gates, because such gates are always built with curved backs and it is not practicable to employ adjustable diagonal braces to take up the sag at the miter end of the gate due to the weight of the gate. In order to reduce this sag, the practice has been to build water-tight buoyancy chambers in the interior of the gate so as to reduce the weight of the gate in water. Of course, provision must be made to keep these compartments dry and clean. In streams which carry a large amount of sediment these compartments will need cleaning after every freshet. Vertically framed gates can be, and usually are, built rectangular in shape, making it possible to use adjustable diagonals with which to take up the sag. Buoyancy chambers are not required and the expense incident to their care and maintenance is avoided.

2. There are fewer members in a vertical frame gate and, therefore, less area to be protected by painting and cleaning.

3. There are fewer large horizontal members to collect silt in a vertical frame gate and, therefore, less expense incurred in its removal.

4. The interior of a vertical frame gate is much more easily accessible on account of the absence of a large number of small cells into which a horizontal frame gate is divided by the many intersecting horizontal and vertical members.

The above indicates that the cost of *maintenance* of vertically framed gates will be less than for the other class.

We have still to consider comparative ease and reliability of *operation*. To dispose of this point, in the absence of examples of direct comparison, impossible because of the nonexistence to date of very large vertically framed gates, I venture the assertion that the latter class will not suffer by comparison when such can be made. I base this assertion on knowledge of the successful working of this type of gate on the small locks of the Kentucky River, and on my inability to discover any reason which would prevent the successful application of the principle to larger lock gates.

All the foregoing seems to point to the desirability of using ver-

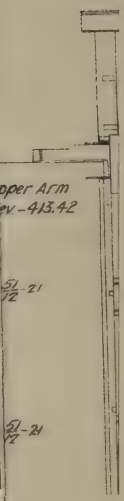
tical framing for miter gates whenever the system will not increase the weight and, considering the greater simplicity of construction and the equal if not greater efficiency, the use of vertical framing appears to be justified even if it should result in a slightly heavier gate.

There is a very marked advantage in favor of the vertical framing in wide locks of low lift, such as the locks on the Ohio River. These locks are all 110 feet wide and, except at Louisville, the lifts do not exceed 9 feet. To provide 11 feet on the lower sill and with a 9-foot lift, the gates are to be 22 feet high, which will allow a freeboard of 2 feet. The gate leaves are about 62 feet long and the ratio of height to length is, therefore, about 35:100. It was on account of the low ratio of height to length that it was formerly considered impracticable to use mitering lock gates supported entirely at one end like a swinging door. Originally it was planned that the Ohio River locks should be 78 feet wide, because, "The lock proposed was believed to be as wide as it was practicable to make it with miter gates of such limited height."*

This width of lock was opposed by the river interests and led to the design by Maj. Wm. E. Merrill, Corps of Engineers, of a new type of gate commonly known as the rolling gate. This gate somewhat resembles "a railroad car which, when out of use, is housed in a recess in the bank and when needed is run across the lock, thus shutting it up and acting as a gate."*

The recesses in which the gates are housed when out of use often fill up with silt during high water. On account of the lack of space within the recesses and the presence of the gate therein, the removal of this deposit is tedious and costly. At Ohio River Lock No. 37, 12 miles below Cincinnati, in the spring of 1913 it was necessary to remove 2,500 cubic yards of silt from the recesses before the gates could be be moved. This required twenty-eight days, part of the time working both day and night, and cost $2,600. On account of previous trouble from the same source, a bulkhead had been built across the entrance to one of the recesses, but in spite of this and the covers which are used over the tops of the recesses, the amount of silt in the recess was about the same as found in the other recess which had no bulkhead. This is, of

*Quoted from Special Report of Board of Engineers on Proposed Movable Dam at Davis Island, Report of the Chief of Engineers, U. S. Army, 1878, page 804, et seq.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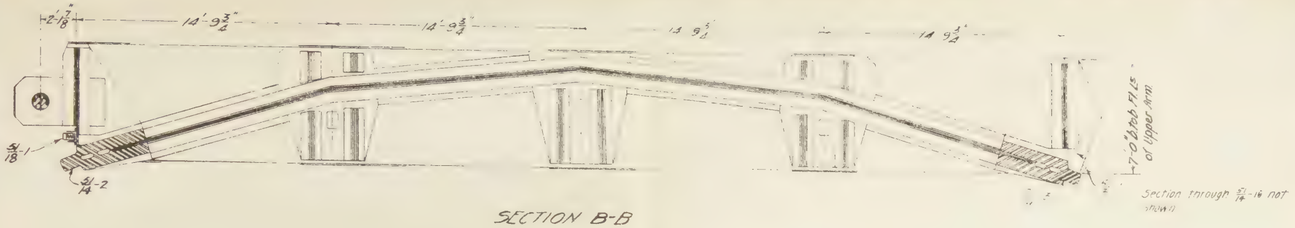
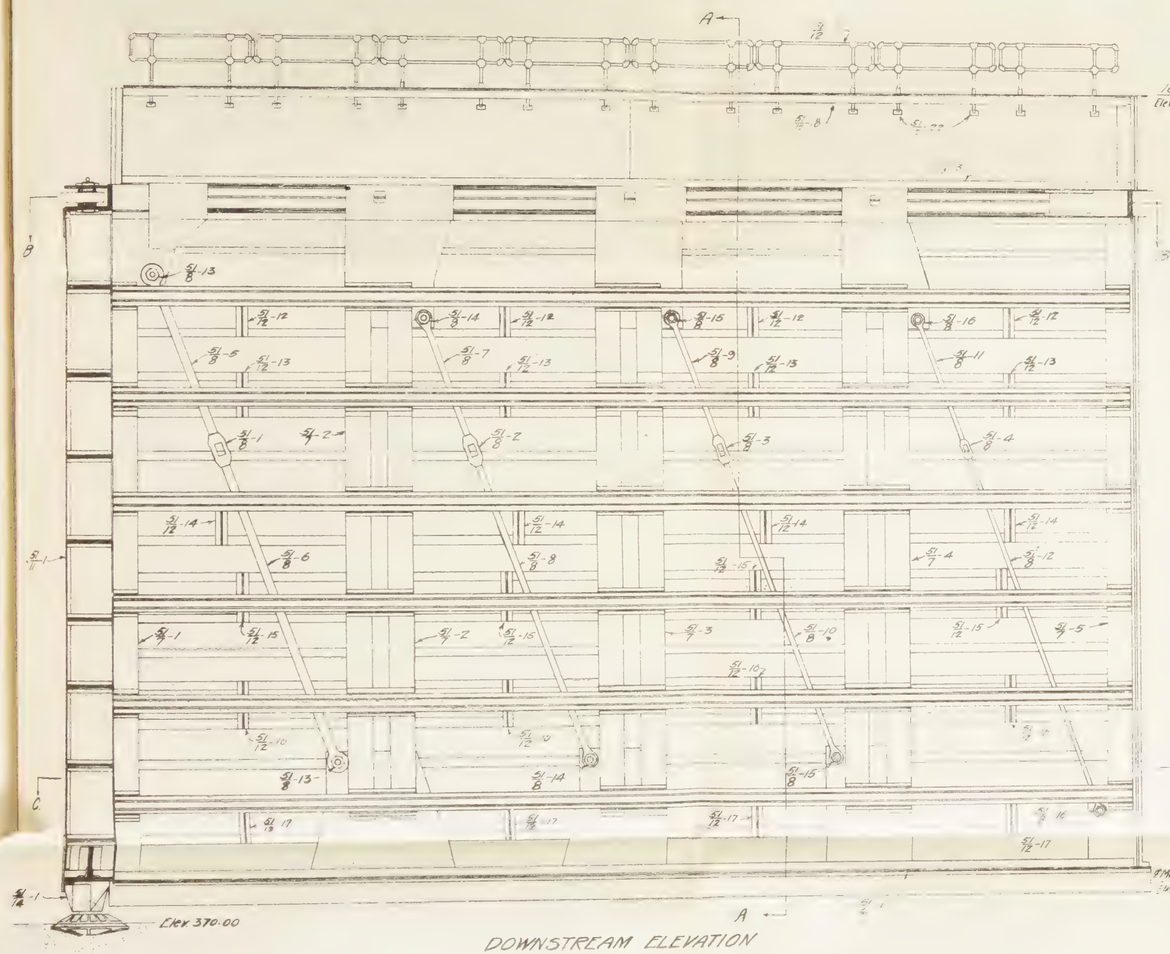
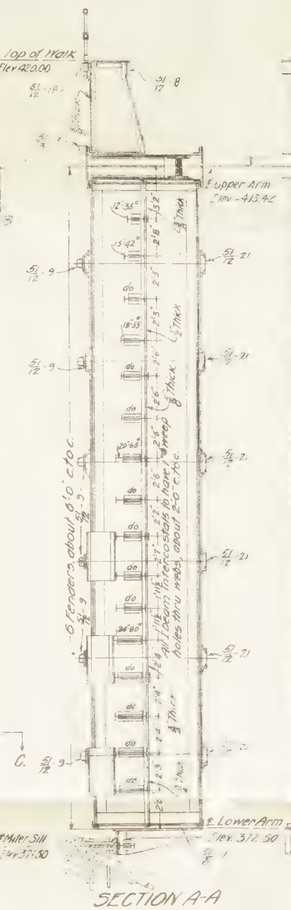
WAR DEPARTMENT

CORPS OF ENGINEERS. U. S. ARMY

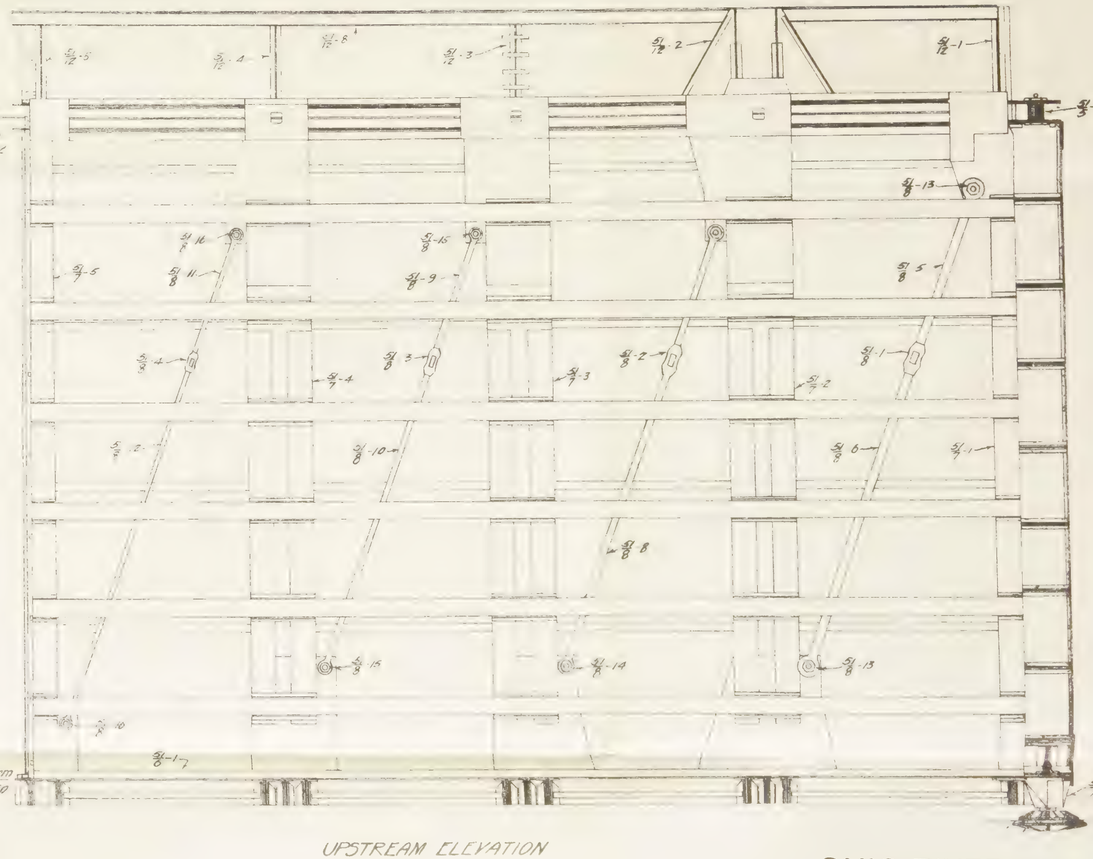


SECTION B-B

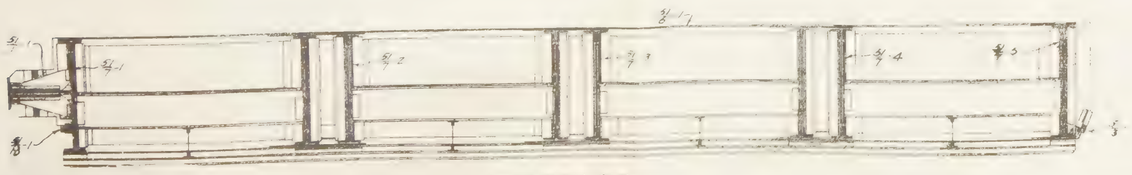
PLAN

DOWNSTREAM ELEVATION



SECTION A-A



UPSTREAM ELEVATION



SECTION C-C

OHIO RIVER
LOCK AND DAM NO. 41
LOCK GATES
LOWER GATE--NORTH LEAF

GENERAL DRAWING

PREPARED UNDER DIRECTION OF THE
OHIO RIVER BOARD OF ENGINEERS ON LOCKS & DAMS

Fig. 6.

course, a serious trouble and it was early recognized that the mitering gate would be better, if practicable, because the recesses for a mitering gate are merely slight depressions in the wall and, therefore, do not collect a large amount of silt, and what does collect is accessible and is easily removed. The doubt has probably been as to whether it would be practicable to build a gate with so low a ratio of height to length.

The upper gate for the new lock which is being built at Louisville, Ky. (Fig. 5), is somewhat similar in dimensions to the other Ohio River lock gates, being 25 feet high and for a lock 110 feet wide. Here it was, for special reasons, decided to use a mitering gate and the design shown in Fig. 5 was worked out. On account of similarity in dimensions it appeared that this design would be satisfactory for the other Ohio River locks. In studying the problem, both vertically and horizontally framed gates were designed, and it was found that the former type could be built for about half the cost of the latter.

The principal question in regard to this gate was whether it would hang symmetrically on its hinges. As shown in Fig. 5, the gate is essentially a cantilever truss, the top girder corresponding to the top chord, and the vertical girders being the compression posts and the eye bar diagonals being the tension members. It is apparent from the design that if there should be any sag or warp in the gate, it could easily be corrected by means of the diagonals. Although this gate is 25 feet high and some of the other Ohio River gates would be only about 17 feet high, no reason could be seen why the same principle could not be applied to the gates of the smaller height. With horizontal framing, however, the problem would be more difficult on account of the extra weight and the difficulty of applying adjustable diagonals to this type.

In view of the above, the vertical frame mitering type has been approved for all Ohio River locks which are to be built in the future, or where construction is not so far advanced that the new type can not be used. The adoption of the new type will result in a large annual saving, and it is not certain that a satisfactory horizontally framed gate could have been designed for these conditions. Vertical framing, therefore, seems to be the proper solution for this particular lock gate problem.

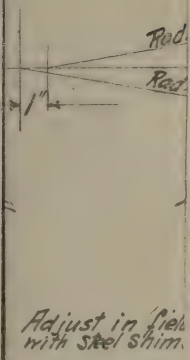
To illustrate the adaptability of the vertically framed gate to varying ratios of height to length, three designs are shown in Figs. 5, 6 and 9. All of these gates are for locks 110 feet wide.

Fig. 5 shows the upper gate for the new lock at Louisville, Ky. The gate is 25 feet high over all, but the sheathing or water-tight portion of the gate is only 22.5 feet high, the part of the gate above the top of the sheathing being merely a superstructure added to support the foot walk at the elevation of the top of the wall, which is 25 feet high. In effect, therefore, the gate is 22.5 feet high. The length is about 62 feet and the ratio of height to length is therefore 36 : 100. The loading assumed for this gate is the most severe possible; that is, water at the top of the sheathing and no water on the downstream side.

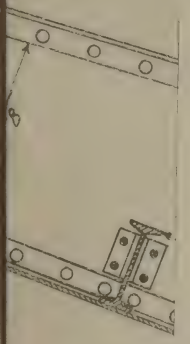
The sheathing is located in the interior of the gate. By this arrangement that part of the bottom of the gate which is on the downstream side of the sheathing will be subjected to upward pressure. By moving the sheathing upstream, the upward pressure would be increased. It is desirable that the sheathing be as near as possible to the center line of the gate, for then the whole structure is symmetrical and will hang evenly on its hinges. But it was not possible with this design to locate the sheathing at the center of the gate, because the upward pressure on half of the bottom area of the gate would have exceeded the weight of the gate. With the location shown, the upward pressure does not quite balance the weight and the sheathing is about as far upstream as it would be permissible to locate it, without providing brackets under the gate so as to allow the gate to overhang the sill, thus reducing the upward pressure. The use of brackets for this purpose is feasible, but was not considered necessary or desirable in this case.

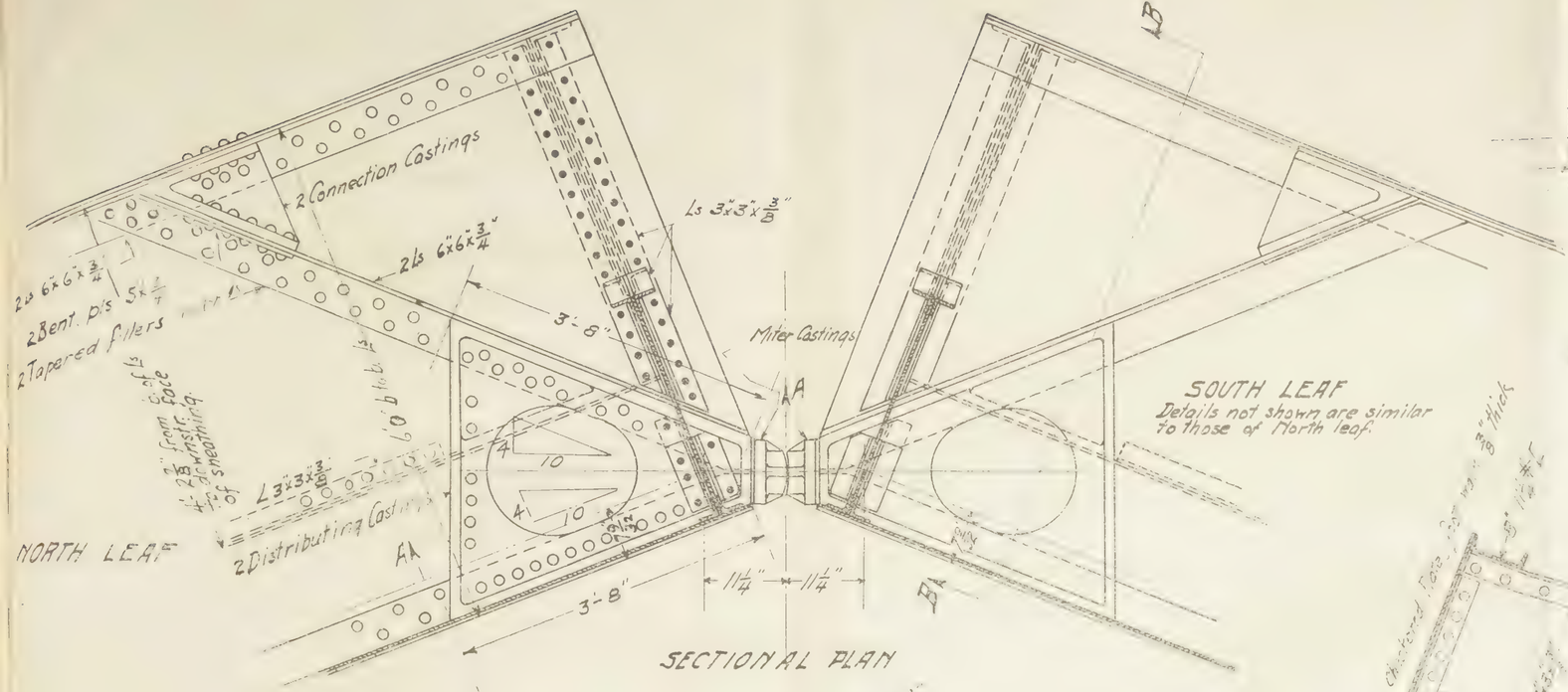
The pressure on the sheathing is carried into the vertical girders through horizontal beams. It would have been possible to space the vertical girders closer together, thereby reducing the loads on the verticals and at the same time obviating the necessity for the horizontals; but investigation showed that the arrangement adopted is more economical in weight. Furthermore, it was believed that the gate would be stiffer and resist better the strains due to operation if the vertical girders were of a depth equal to the width of the gate, and this leads to wide spacing so as to make the loads correspond with the strength of the girders.

The verticals are plate girders designed according to usual practice. They are connected at the bottom by a horizontal girder whose only function is to hold the vertical girders together, forming a water-tight contact with the sill. This member does not take any calculated strain except the upward pressure above men-



Adjust in field
with steel shims





SOUTH LEAF
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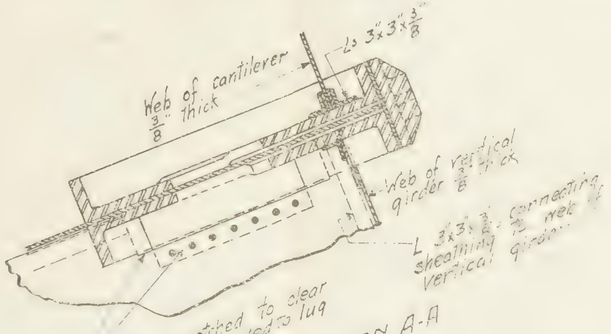
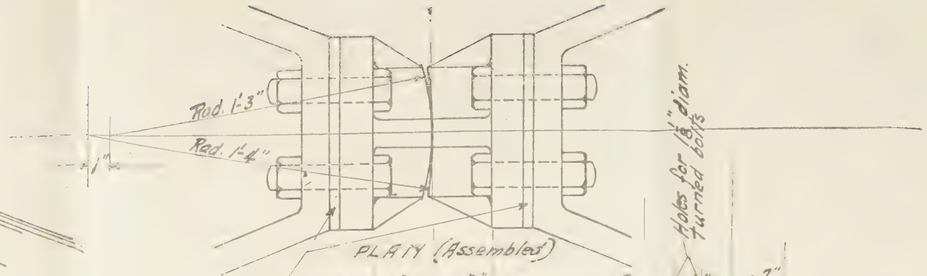
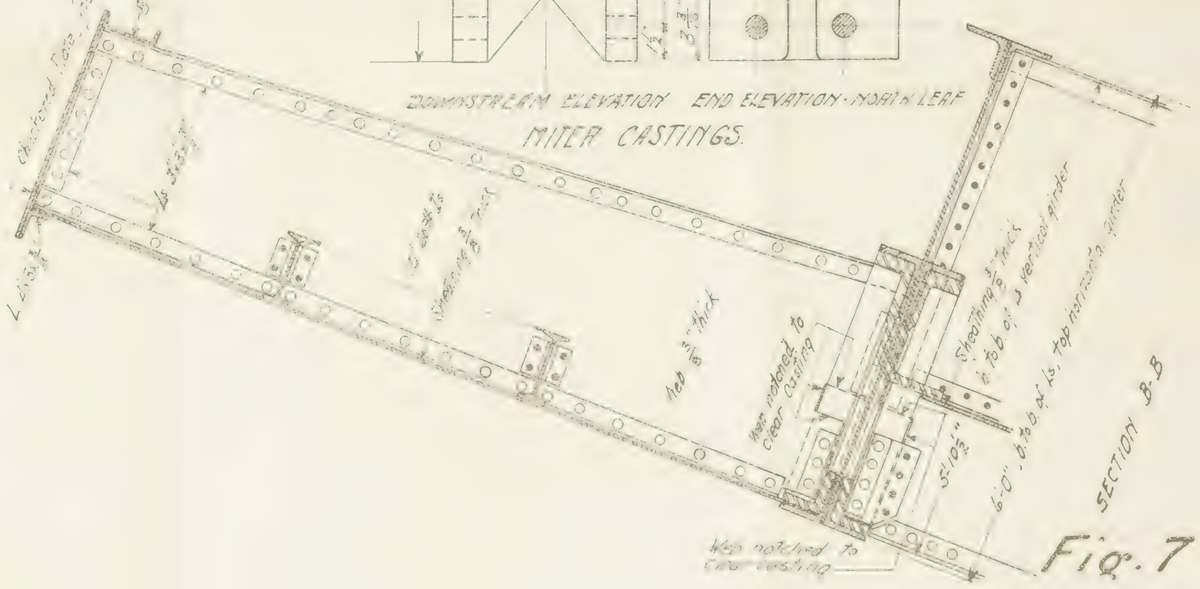


SECTION A-A



DOWNSTREAM ELEVATION END ELEVATION - NORTH LEAF

MITER CASTINGS.



SECTION B-B

Fig. 7

tioned. The lower ends of the vertical girders are supported by the sill, which is anchored to the floor of the lock. The upper ends of the verticals are connected to and supported by a horizontal girder, which is in turn supported at one end by the corresponding girder of the opposite gate leaf and at the other end by the lock wall.

Special provision is made for the large thrust at the ends of the top girder. The point of application of this thrust is confined within narrow limits, because if this point is variable in position it results in varying stresses throughout the entire length of the girder. For instance, if the point of contact is upstream from its theoretical position the stress in the upstream flange of the girder throughout its entire length is increased. Since each flange must be designed for the most dangerous position of the point of contact, it is clear that it is advisable to confine this point of contact within as narrow limits as possible.

Fig. 7 shows the arrangement adopted for the new guard gate at Louisville (Fig. 9). The convex face of the casting on one of the leaves fits into a concave recess in a similar casting on the other leaf. The convex and the concave curves are of slightly different radii, the latter being the greater. The initial contact, therefore, is along an element of a cylinder. The necessary bearing area is, of course, developed by yielding of the metal, just as is the case when a weight is supported by rollers. The advantage of using two different radii is that a slight variation in the positions of the leaves when mitering will cause a smaller variation in the location of the center of pressure than would be the case if the two meeting faces were in contact throughout their entire width.

The gate shown in Fig. 6 is also for a lock 110 feet wide, but the height is 48 feet. The ratio of height to length is, therefore, 77 : 100. This is a design which was proposed as the lower gate for the new lock at Louisville, Ky., but was rejected by the Chief of Engineers. At about the time of the rejection of this design a horizontal arch gate was designed under the direction of Mr. Hugh L. Cooper, M. Am. Soc. C. E., for the new lock at Keokuk, Iowa. The dimensions and conditions were almost identical with those at Louisville, and it appeared that the Keokuk gate would be satisfactory, so the design shown in Fig. 8 was worked out. This design is practically a copy of the Keokuk gate, the only differences being those which local conditions seemed to make necessary.

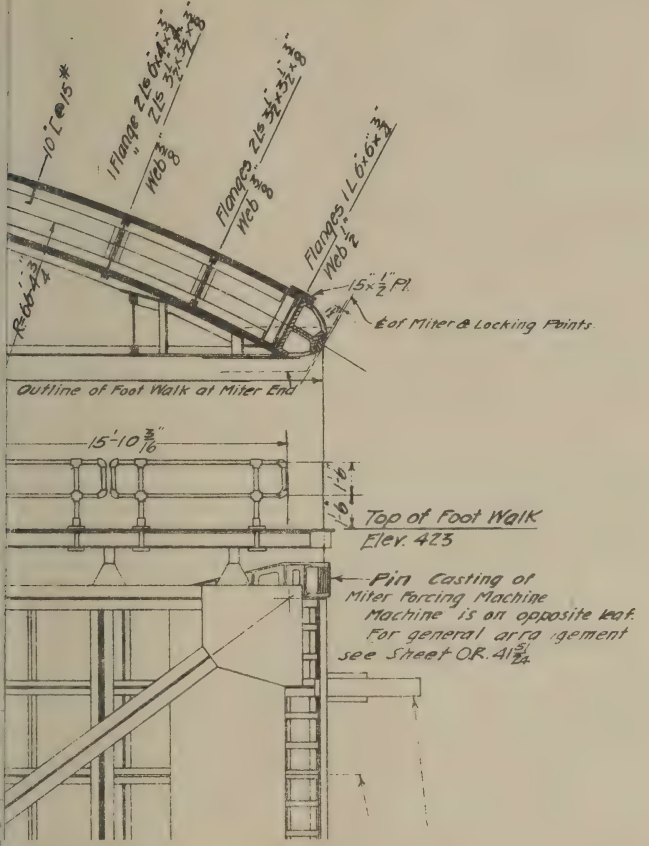
The vertical frame and horizontal arch designs having been made

for the same conditions, an excellent opportunity exists to make a comparison between the relative weights of the two types. Furthermore, there has been built for the Panama Locks a gate almost identical in size (lock width 110 feet, height of gate 47 feet 4 inches) with horizontal girders instead of arches, and we can thus compare vertical frames, horizontal arches and horizontal girders. The comparison is as follows: The horizontal arch gate and the vertically framed gate each weigh about 500,000 pounds per leaf. The horizontal girder gate weighs 750,000 pounds per leaf. There is no question but that the construction is much more simple in the vertically framed gate than in either of the other two.

The vertical frame gate shown in Fig. 6 is somewhat similar in construction to the vertical frame gate recently erected at St. Paul for a lock 80 feet wide. This gate is described in *Engineering News*, July 2, 1914, page 1. The idea of using the arch strut to carry the loads at the top was obtained from the design of the St. Paul gate. With the heavy gate shown in Fig. 6 (approximately 500,000 pounds per leaf) it was decided to locate the sheeting on the center line of the gate to avoid warping tendency. The gate is designed to overhang the sill somewhat, so as to reduce the upward pressure, which would otherwise exceed the weight of the gate. The sill reaction is carried by brackets, as shown; the tension being carried by bolts so as to avoid rivets in tension.

Another design of vertically framed gate is shown in Fig. 9. This is the guard gate for the new lock now being built at Louisville. It is 44 feet high and for a lock 110 feet wide, each leaf being about 62 feet long. The ratio of height to length is 71 : 100.

The principal use of this gate will be to stop the flow through the lock during high water. At such times there is only 2 or 3 feet difference in the elevation of the upper and lower pools and, therefore, the loads on the gate are not very great. Another use to which it will be put will be to act as a coffer when it is desired to have access to the upper gate. In this case there can never be a head greater than 22 feet on the upstream side, and this head, of course, would act only on the lower part of the gate. For these reasons this gate will never be acted upon by loads as great as would ordinarily be the case with a gate of this size. On this account, and also on account of the use of vertical framing, it is a very light structure, considering its size. The weight will be about 180,000 pounds per leaf. Except for greater height, this gate is very similar


10 L @ 15'-0"
1 Flange 2 L 6 x 4 x 3/8
2 L 3 1/2 x 3 1/2 x 3/8
Web 3/8"
Flanges 2 L 3 1/2 x 3 1/2 x 3/8
Web 3/8"
Flanges 1 L 6 x 3 1/2
Web 1/2"
Rods 4 1/2"
15" x 1/2" Pl.
Leaf Miter & Locking Points
Outline of Foot Walk at Miter End
15'-10 3/16"
1'-0"
1'-0"
Top of Foot Walk
Elev. 423
Pin Casting of
Miter Forcing Machine
Machine is on opposite leaf.
For general arrangement
see Sheet O.R. 41 3/4

SECTION A-A

66'-4¾" center to center of bearings

Outline of Foot Walk at Miter End

Top of Foot Walk Elev. 423

Pin Casting of Miter Forcing Machine. Machine is on opposite leaf. For general arrangement see Sheet OR. 41 3/24

Miter Guard here on opposite Gate Leaf

Miter Guard here For Detail see Sheet OR 41 5/16

Leaf Miter & Locking Points

Operating Strut Connection For detail see OR. 41 3/24

Gusset Pls. ½ unless shown otherwise

SECTION B-B

DOWNSTREAM ELEVATION

APPROVED BY THE OHIO RIVER BOARD, JANUARY 10, 1914.

A. C. Schaller
Captain Corps of Engineers,
Recorder.

APPROVED BY THE CHIEF OF ENGINEERS U. S. A. MARCH 14, 1914.

OHIO RIVER
LOCK AND DAM NO. 41.
LOWER GATE
GENERAL DRAWING

PREPARED UNDER DIRECTION OF THE OHIO RIVER BOARD OF ENGINEERS ON LOCKS & DAMS

Fig. 8.

in construction to the upper gate for the same lock shown in Fig. 5.

These examples of vertical framing for large lock gates are shown to demonstrate the practicability and economy of this type of construction, for varying ratios of height to length. Considering the points discussed, the opinion is here stated that, in general, vertical framing is superior to horizontal framing wherever the height of the gate does not exceed its length. But a survey of present-day practice indicates that this opinion is not yet generally accepted by lock gate designers. In this country there are several locks 100 feet or more wide, but none of them is equipped with vertically framed gates. The largest gate of the vertically framed type is that in the 80-foot lock at St. Paul, to which reference has been made. There the ratio of height to length is a little over 1 : 1, and it is claimed that a saving was made over the horizontally framed type. Opposed to this solitary example, we have the horizontal arch gates for the 100-foot lock at Sault Ste. Marie, whose ratios of height to length vary from 45 : 100 to 77.5 : 100; the horizontal arch gates for the 90-foot lock on the Cascades Canal with ratios varying from 67 : 100 to 86 : 100, and the horizontal arch gate for the 110-foot lock at Keokuk, Iowa, whose ratio of height to length is about 77.5 : 100.

The prevailing opinion in favor of horizontally framed gates suggests that there may be some valid objection to vertical framing. In searching for some such objection the writer has studied existing reports and literature as thoroughly as facilities available would permit, but has not yet found a good reason for rejecting vertical framing where the dimensions were such as to make this type more economical.

An objection occasionally advanced against vertical framing is the heavy load on the sill, which might be dangerous should the character of the foundation be not all that could be desired. This objection can easily be overcome, however, at very little, if any, extra cost. If the foundation under the sill is not capable of carrying the loads, it is necessary only so to design the sill that it will carry the load by arch action into the walls, in which case the wall will take exactly the same total load as in the case of a horizontally framed gate. The only difference is in the arrangement of loads: with a vertically framed gate the walls receive a part of their load from a masonry arch at a single point, while with a horizontally

framed gate the walls receive all their load from steel arches or struts at several points.

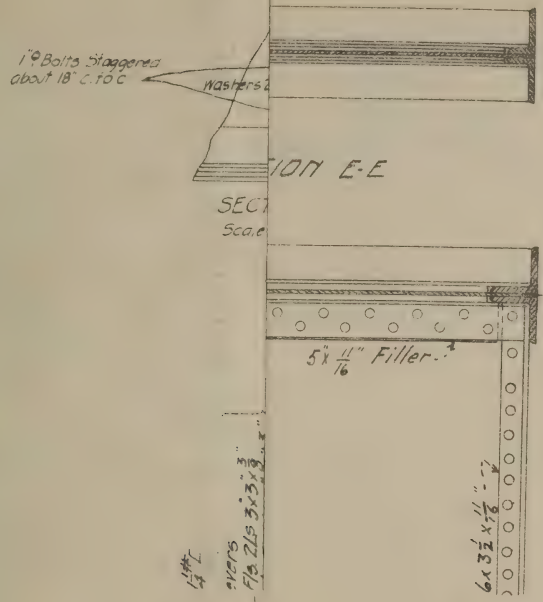
For either type of gate the sill foundation must be sufficiently sound to prevent excessive leakage, and the tendency to leak under the sill is, of course, no greater with one type of gate than with another.

For the Ohio River locks vertically framed gates are now to be used in place of the rolling gates. In this case the sill reaction is the same for either type, because the rolling gates carry their loads through vertical frames. But the conditions, so far as the sill loads are concerned, are more favorable with mitering gates, because the sill masonry is arched with mitering gates and straight with the rolling gates.

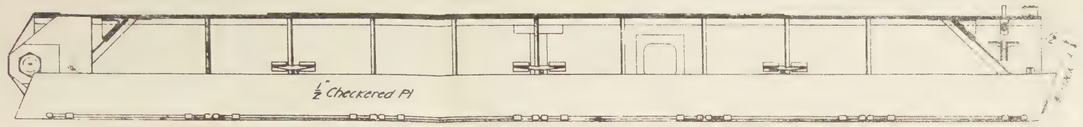
It has been suggested that a horizontally framed gate stands less chance of being wrecked by being struck by a boat. The contention is that with horizontal framing, part of the gate might be damaged and still enough of the gate remain mitered to hold back most of the water, thus averting disaster to other gates or shipping below the gates struck. Judging from the number of accidents that have occurred recently in locks equipped with horizontally framed gates, it is evident that this type of framing does not prevent such accidents nor is there any evidence that the consequences would have been more disastrous if the gates had been vertically framed.

No lock gate has ever been built which would withstand a blow from a heavy vessel going at high speed. The majority of accidents seem to occur from a vessel striking the gate on the downstream side. Here the damage is not confined to the gate which has been struck, but consists also of the effects of the rush of water which takes place when the gates are unmitered by the impact. It would be just as easy to unmiter one type of gate as another by a blow from the downstream side. Immediately after the gates are unmitered the movement upstream caused by the collision is arrested if there is any head acting on the gates and, whatever may be the type of construction, the gates will then be forced downstream over the miter sill and be torn from their hinges.

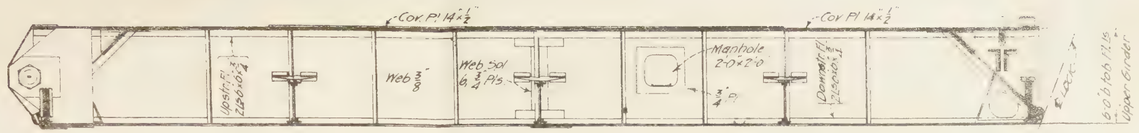
Should a vertically framed gate be struck on the upstream side, the upper horizontal must first be wrecked or knocked out of miter before the gate would fail. It would be possible to use a locking device which would reduce the possibility of the leaves being separated by impact, and such a device on a vertically framed gate would be much more effective than a similar device on a horizontal-


1"ø Bolts Staggered
about 18" C. to C
Washers
SECTION E-E
Scale
5" x 11/16" Filler
6 x 3½ x 11/16"
6'-0" to fl. ls.
Upper Girder

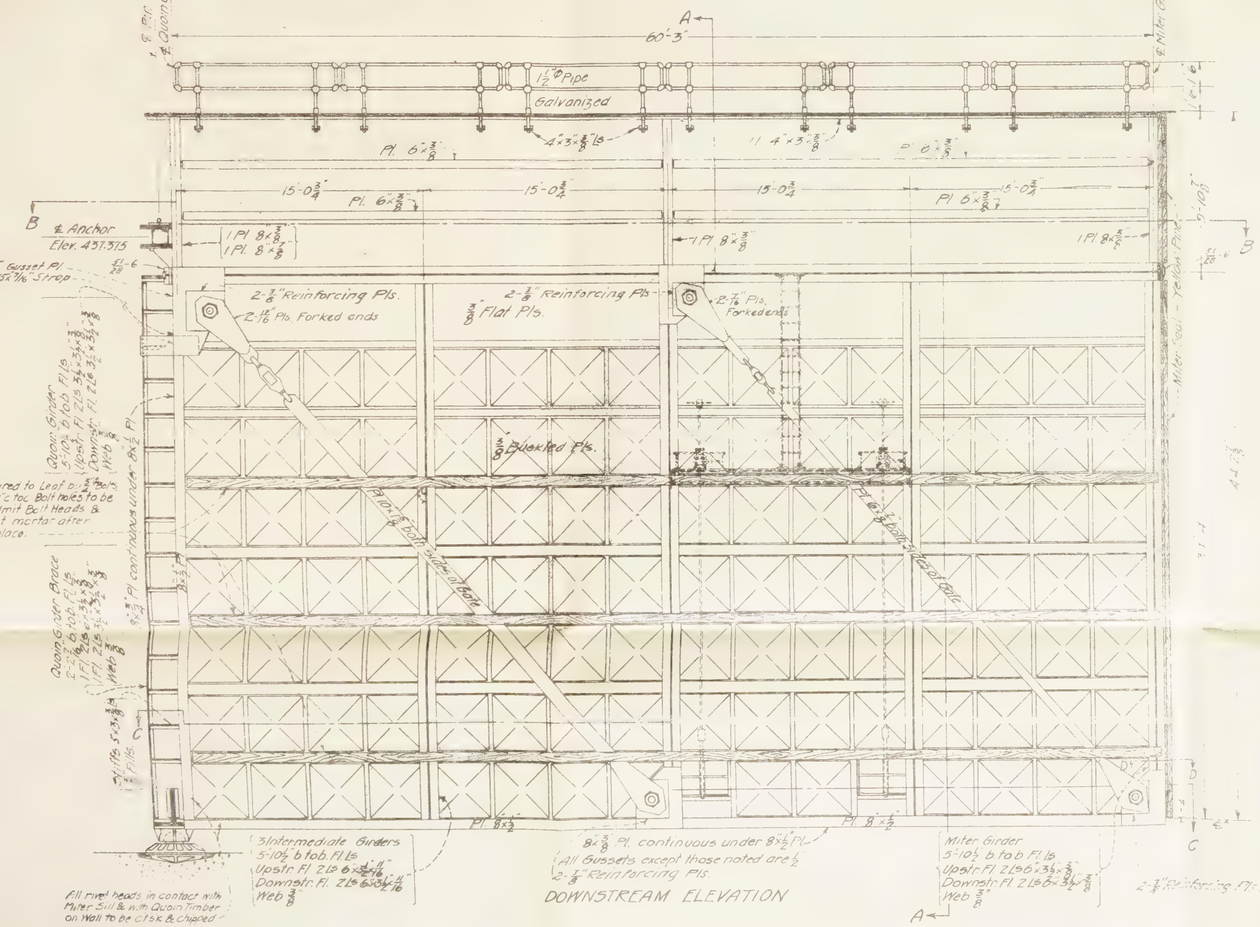
WAR DEPARTMENT



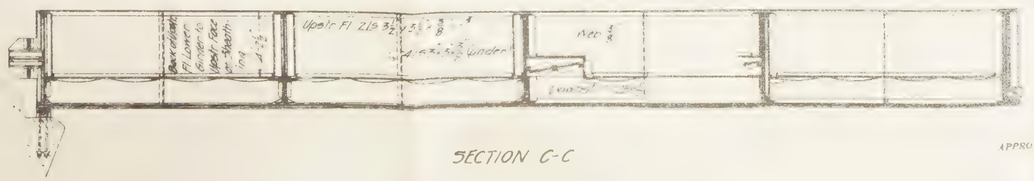
PLAN



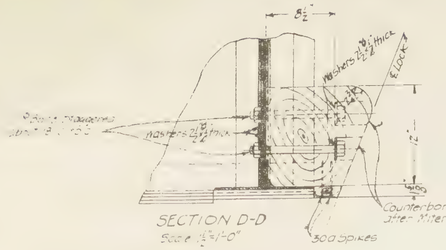
SECTION B-B


Quoin Girder
A
60'-3"
1½" ϕ Pipe Galvanized
4"x3"x⅜" Ls
Pl 6'-3⅜"
11 4"x3"x⅜" Ls
Pl 8'-3⅜"
15'-0¾"
Pl. 6"x⅜"
1 Pl 8"x⅜"
1 Pl 8"x⅝"
Yellow Pine 5'-10½"
B
℄ Anchor Elev. 437.315
6¼" Fill under 1½" Gusset Pl
5 15/16" Fill under 1½" Strap
2-⅞" Reinforcing Pls.
2-1⅛ Pls. Forked ends
2-⅜" Reinforcing Pls
⅜" Flat Pls.
2-⅞" Pls. Forkedends
⅝" Bucklod Pls.
Miter Sill
Fenders to be secured to Leaf by 1" ϕ bolts staggered about 3'-3" c to c. Bolt holes to be counterbored to admit Bolt Heads & filled with cement mortar after Fenders are in place.
Quoin Girder 5'-10½" b to b. Fl Ls
Upstr. Fl 2 Ls 6"x6"x⅝"
Downstr. Fl 2 Ls 6"x6"x⅝"
Web ⅜"
Quoins 6 per Brace
Stiffs 5"x3"x⅜" Ls
Pl 8"x½"
All Fillet heads in contact with Miter Sill & with Quoin Timber on Wall to be Cut & Chipped
3 Intermediate Girders
5'-10½" b to b. Fl Ls
Upstr. Fl 2 Ls 6"x3½"x⅜"
Downstr. Fl 2 Ls 6"x3½"x⅜"
Web ⅜"
8"x⅜" Pl. continuous under 8"x½" Pl.
All Gussets except those noted are ½
2-⅞" Reinforcing Pls.
Miter Girder
5'-10½" b to b. Fl Ls
Upstr. Fl 2 Ls 6"x3½"x⅜"
Downstr. Fl 2 Ls 6"x3½"x⅜"
Web ⅜"
2-⅞" Reinforcing Pls.
Pl 8"x½"
C
A

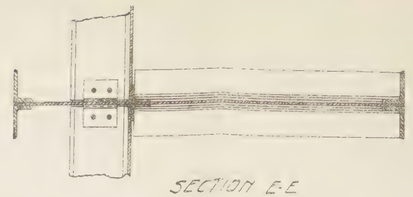
DOWNSTREAM ELEVATION



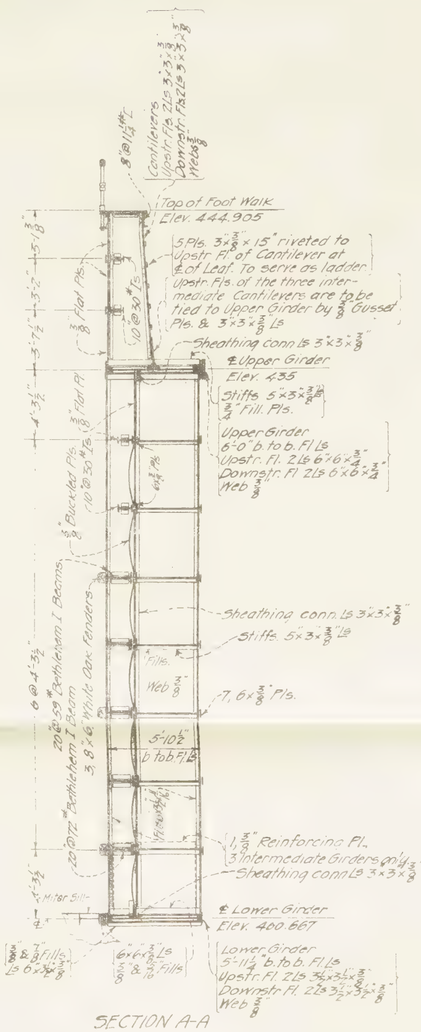
SECTION C-C


8½"
Washers 2½"x2½"x⅜" thick
E Lock
Steel Masonry Anchor 3'-10"
Washers 2½"x2½"x⅜" thick
300 Spikes
Counterbore 2¾" ϕ & fill with cement mortar. Miter Sill Seal is in place

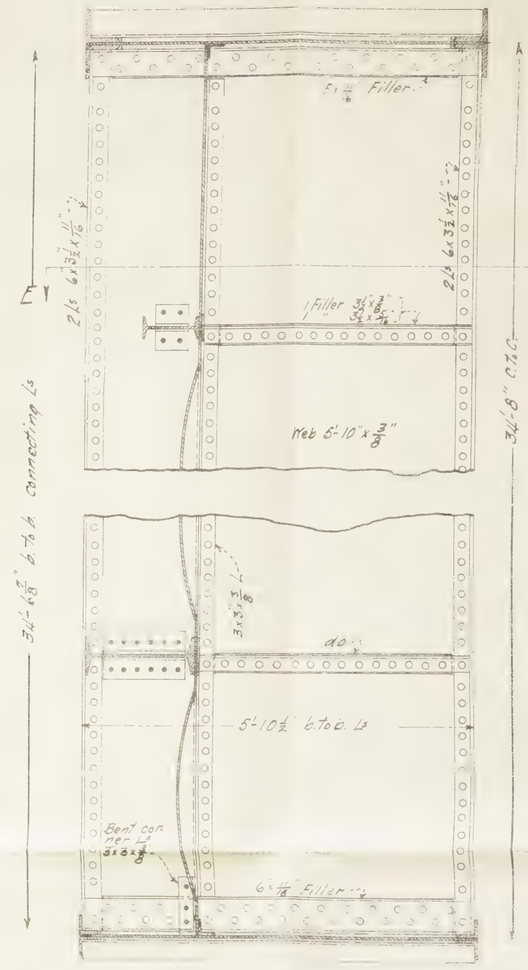
SECTION D-D
Scale 1½"=1'-0"



SECTION E-E


Cantilevers Upstr Fl 2 Ls 3"x3"x⅜" Downstr Fl 2 Ls 3"x3"x⅜" Web ⅜"
Top of Foot Walk Elev. 444.905
5 Pls. 3"x⅜"x15" riveted to Upstr Fl of Cantilever at ℄ of Leaf. To serve as ladder. Upstr Fls. of the three intermediate Cantilevers are to be tied to Upper Girder by ⅜" Gusset Pls. & 3"x3"x⅜" Ls
Sheathing conn Ls 3"x3"x⅜"
℄ Upper Girder Elev. 435
Stiffs 5"x3"x⅜"
⅜" Fill Pls.
Upper Girder 6'-0" b to b. Fl Ls
Upstr. Fl 2 Ls 6"x6"x⅜"
Downstr. Fl 2 Ls 6"x6"x⅜"
Web ⅜"
Sheathing conn Ls 3"x3"x⅜"
Stiffs 5"x3"x⅜" Ls
Fills
Web ⅜"
7, 6"x⅜" Pls.
5'-10½" b to b Fl Ls
1 ⅜" Reinforcing Fl.
3 Intermediate Girders only ½"
Sheathing conn Ls 3"x3"x⅜"
Miter Sill
℄ Lower Girder Elev. 400.667
Lower Girder 5'-11¼" b to b. Fl Ls
Upstr. Fl 2 Ls 3½"x3½"x⅜"
Downstr Fl 2 Ls 3½"x3½"x⅜"
Web ⅜"
⅜" Fills
6"x6"x⅜" Ls
⅜" Fills
6 @ 4'-3½" = 20'-7½" Bethlehem I Beams, 3.8 x 6 Wide Ox Fenders
Bucklod Pls. ⅜" Flat Pl
Flat Pls.
Miter Sill

SECTION A-A



TYPICAL VERTICAL GIRDER.

OHIO RIVER
LOCK AND DAM NO. 41.
GUARD GATE
GENERAL DRAWING

PREPARED UNDER DIRECTION OF THE
OHIO RIVER BOARD OF ENGINEERS ON LOCKS & DAMS

APPROVED BY THE OHIO RIVER BOARD, FEBRUARY 17, 1914
Captain, Corps of Engineers,
Recorder.

APPROVED BY THE CHIEF OF ENGINEERS, U. S. A., APRIL 23 1914

Fig. 9.

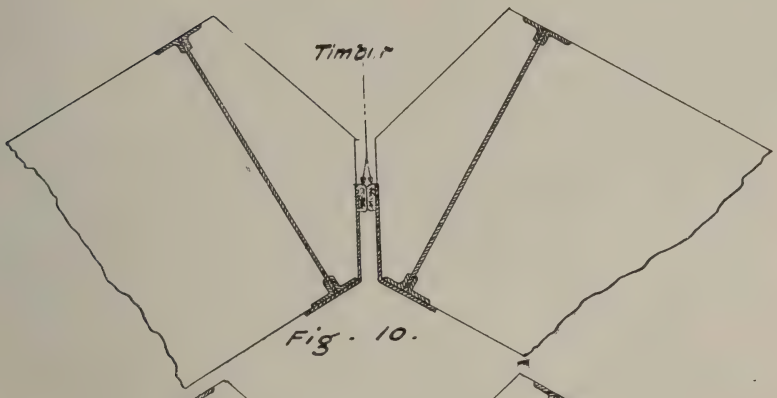


Fig. 10.

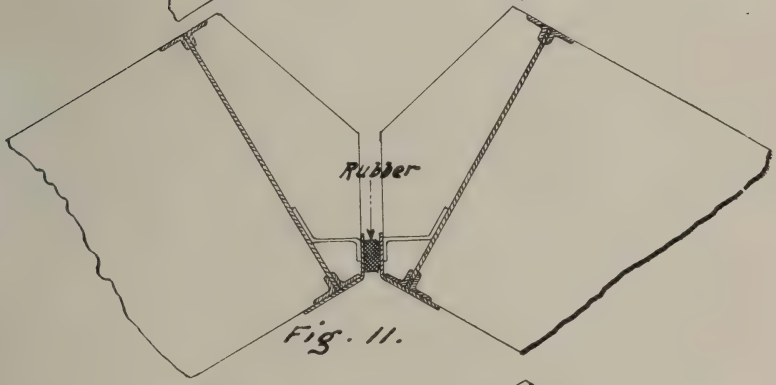


Fig. 11.

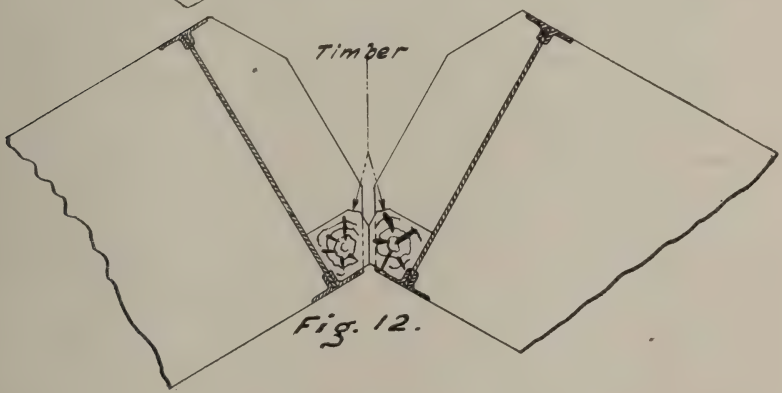


Fig. 12.

ly framed gate, for in the one case it would be anchored to a very substantial member: viz, the top girder designed to carry one-third of the water pressure, and in the other case it would be anchored to a comparatively light and flexible member which would yield under a much smaller impact than the heavy horizontal girder of a vertically framed gate.

One feature in the design of a vertically framed lock gate, which at first sight might seem troublesome, is the flexible contact at the quoin and miter posts, which will prevent leakage of water and at the same time not develop excessive thrust in the gate below the top girder. This problem, however, is readily solved by the use of some sort of elastic material at the quoin and miter posts. Fig. 10 shows the arrangement used on the vertically framed gate at St. Paul, to which previous reference has been made. It consists of a pair of flexible plates provided with wooden buffers extending from the top to the bottom of the gate. These plates are about ⅜-inch thick and therefore of sufficient flexibility to allow for any reasonable variation in the relative position of the two miter posts. The plates will yield long before they will transmit any dangerous thrust into the gate, but at the same time they are strong enough to resist the water pressure on them. The single objection to this scheme is that the plates are so fragile that in case an obstruction lodges between them they will probably be bent out of shape and have to be straightened out again before they will act properly.

Fig. 11 shows a scheme which is free from the objection which is stated above. It consists of a rubber buffer which makes a contact with a braced plate on the opposite leaf. So far as known this scheme has never been tried, but no reason is seen why it should not work. On small gates the common practice is to use wooden blocks framed into the miter posts, as shown in Fig. 12. This is a comparatively simple arrangement and should work satisfactorily in most cases. It is to be noted that all the arch thrust is to be taken by the top girder; that there are no horizontal members intermediate between the miter sill and the top girder and that in case the miter posts meet before the miter ends of the top girder meet, something will yield until these two top members do come in contact. For this reason the wooden miter posts should be planed down so that they will not quite touch when the top girders miter. Then, when the thrust shortens the top members, the two timbers will come together, holding back the water but not unduly straining the gate.

Other objections to vertical framing, so far as can be ascertained from official reports on the subject, are that for the cases under consideration the vertical framing would give heavier gates. But this conclusion is generally arrived at by assuming that all the gates of a given project should be of the same design; that is, that it would not be advisable to use the horizontal framing for a lofty lower gate and vertical framing for a low upper gate, in the same lock. This assumption is not usually justified. Such a condition existed in the lock recently built on the Mississippi River at Keokuk. The lower gate is the mitering horizontal arch type, similar to that shown in Fig. 8, while the upper gate, which is much lower, is not even a mitering gate. It is a vertical lift structure and does not in the least resemble the lower gate.

The real objection to vertical framing is that it is still, to a large extent, a novelty on large locks. The use of this type in the lock at St. Paul is an instance of departure from standard practice, for it is the largest vertical frame gate ever built in this country. The new gates at Louisville, when built, will be larger yet. It is believed that these examples will demonstrate the advantages of vertical framing for large lock gates and that this type of construction will be more generally adopted when it is better understood.

Relative Duties of Federal and of State or Municipal Officials in Port Development.*

BY

Col. W. M. BLACK,
Corps of Engineers, U. S. A.

To understand the duties of Federal officials in questions of this class, a knowledge of the responsibilities and powers of the General Government is necessary.

Remembering the burdens caused in Colonial days by the restrictions then placed on commerce by law, the framers of the Constitution wisely included in the powers surrendered by the States to the General Government the entire control of interstate commerce, as enumerated in the first and third articles.

Early in the Nation's life, the generalities of the fundamental law had to be interpreted and applied to particulars by the Congress and the Justices of the Supreme Court of the United States and providentially, as it now seems, the Justices of that Court have been men of wide vision. From the beginning, both the laws enacted and the interpretation made of those laws have been in harmony, and have consistently ruled that the powers of the United States over the instruments of interstate commerce are supreme. Late decisions have simply served to emphasize the wide extent of the field of action of those powers.

The local laws of the various states relating to the navigable waters differed in accordance with the origin from which sprung the original settlers. The thirteen Colonies derived their laws and ideals of law from England. In the States formed from the territories acquired by purchase, or as the results of a successful war, the Spanish law prevailed. In all, the control of the navigable waters had been vested in the Crown as a trust for all of the people and the

*Paper read by Col. W. M. Black, Corps of Engineers, U. S. A., before the Convention of the National Association of Port Authorities, at Baltimore, Md., September 8, 1914.

points of difference in the laws related mainly to property boundaries and riparian rights. In the transfer of sovereignty, this right of control had become a part of the powers of the people; at first in the trust of the separate Colonies, but later surrendered by the States, their successors, to the Nation.

When new States were formed from the acquired territory the provisions of the Constitution were made applicable to them. Thus, the rights of navigation have been made common to all of the people and the laws enacted for the preservation of these rights are national laws enacted by Congress. The several States may and do enact legislation regarding the navigable waters within their limits and such laws are valid in so far as they do not conflict with the laws of the Nation. In many cases, Congress has acted slowly in the exercise of the national powers and even to-day the full extent of these powers has not been covered.

Under the English precedent the navigable waters were limited to those in which the tide ebbs and flows. It was early perceived that such a limitation could not be applied to the waterways of the United States and the Supreme Court had to make a new definition. Perhaps the best and most comprehensive definition is that given by Mr. Justice Field in the Daniel Ball case, 10 Wall., 557, in which he states:

Those rivers must be regarded as public navigable rivers in law which are navigable in fact. And they are navigable in fact when they are used or are susceptible of being used, in their ordinary condition, as highways for commerce, over which trade and travel are or may be conducted in the customary modes of trade or travel on water. And they constitute navigable waters of the United States within the meaning of the acts of Congress, in contradistinction from the navigable waters of the states, when they form in their ordinary condition by themselves, or by uniting with other waters, a continued highway over which commerce is or may be carried on with other states or foreign countries in the customary modes in which such commerce is conducted by water.

In *Gilman* vs. *Phila.*, 3 Wallace, 713, Mr. Justice Swayne, delivering the opinion of the Court, said:

Commerce includes navigation. The power to regulate commerce comprehends the control for that purpose, and to the extent necessary, of all the navigable waters of the United States which are accessible from a State other than those in which they lie. For this purpose they are the public property of the Nation, and subject to all the requisite legislation by Congress. This necessarily includes the power to keep them open and free from any obstructions

to their navigation, interposed by the State or otherwise; to remove such obstructions when they exist, and to provide, by such sanctions as they may deem proper, against the occurrence of the evil and for the punishment of the offenders. For these purposes Congress possesses all the powers which existed in the States before the adoption of the National Constitution, and which have always existed in the Parliament in England.

It is for Congress to determine when its full power shall be brought into activity, and as to the regulations and sanctions which shall be provided.

In *Stockton* vs. *The Baltimore and New York Railroad Co.* (32 Fed. Rep., 9), Mr. Justice Bradley, in rendering the opinion of the Court, in answering the contention that although Congress may have power to construct roads and other means of commercial communication between the States, yet this can only be done with the concurrence and consent of the States in which the structures are made, said:

If this is so then the power of regulation in Congress is not supreme; it depends on the will of the State. We do not concur in this view. We think that the power of Congress is *supreme* over the whole subject, unimpeded and *unembarrassed* by State lines or State law; that in this matter the country is one and the work to be accomplished is national, and that State interests, State jealousies and State prejudices do not require to be consulted. In matters of foreign and interstate commerce there are no States.

Comprehensive as this is, later decisions have given even a wider scope to the powers of the people in the preservation of the navigable waters, as in the case of the *United States* vs. *Rio Grande Dam and Irrigation Co.* (174, U. S. 690), in which the Court said:

Although this power of changing the common law rules as to streams within its dominion undoubtedly belongs to each State, yet two limitations must be recognized: First, that in the absence of specific authority from Congress a State can not by its legislation destroy the right of the United States, as the owner of lands bordering a stream, to the continued flow of its waters; so far at least as may be necessary for the beneficial uses of the Government property. Second, that it is limited by the superior power of the General Government to secure the uninterrupted navigability of all navigable streams within the limits of the United States. In other words, the jurisdiction of the General Government over interstate commerce and its natural highways vests in that Government the right to take all needed measures to preserve the navigability of the navigable water courses of the country even against any state action.

Decisions restricting the power of the States to make grants of submerged lands to private parties or to corporations are found in *Illinois Central R. R. Co.* vs. *Illinois* (146 U. S. 387), and in *The People of the State of California* vs. *Southern Pacific R. R. Co.* (Superior Court of the State of California).

In the administration of the laws relating to navigable waters, the question of the ownership of the submerged lands does not enter. The rights of navigation extend to the high-water mark of tidal waters and to the ordinary water line of nontidal waters, irrespective of such ownership, and the use which an owner of lands submerged by the navigable waters can make of his property must be such as does not interfere with the prior rights of navigation. Naturally, there must be authority vested in some one to decide whether a proposed use of the submerged lands can be permitted. In the acts of March 3, 1899, June 13, 1902, August 18, 1894, May 9, 1900, and March 3, 1905, are found the general laws for the protection of the navigable waters. A general bridge law exists in the act of March 23, 1906, and one relating to dams in the act of June 28, 1910. All constructions in or changes of capacity of navigable waterways are forbidden unless specifically authorized by Congress or in certain cases by the Secretary of War, to whom Congress has deputed a limited discretionary authority.

For example, minor constructions on the shores or in the beds of navigable waterways may be authorized under permits from the Secretary of War. Further, section 11 of the Act of March 3, 1899, provides:

> That where it is made manifest to the Secretary of War that the establishment of harbor lines is essential to the preservation and protection of harbors he may, and is hereby, authorized to cause such lines to be established, beyond which no piers, wharves, bulkheads, or other works shall be extended or deposits made, except under such regulations as may be prescribed from time to time by him.

Harbor lines usually consist of two bounding lines along the shores of a waterway. That nearer the shore marks a limit for solid fill, and is called a bulkhead line. The other marks a limit for open constructions beneath which the water can circulate, and is termed a pierhead line. In certain cases the two are made coincident. In others, only a pierhead or a bulkhead line is established. In some cases the establishment of one or both lines is coupled with the proviso that no construction shall be made between the shore and the line or lines, excepting under a permit from the

Secretary of War in which the nature, extent and location of the structure shall be specified.

The establishment of harbor lines grants no privileges of ownership, but simply signifies that within the limits set by the lines there is no objection made by the United States, as trustee for the people, to the use of these submerged lands for the class of structures authorized. The establishment of harbor lines is in the nature of the issuance of a general permit. Within the areas bounded by the harbor lines the uses of the submerged soil may be still further regulated by State authority.

The lines as established are subject to change where in the opinion of the Secretary of War the needs of navigation so require. (See *Phila. Co.* vs. *Stimson*, 223 U. S., 605.) Further, in a decision dated September 27, 1910, the Attorney General of the United States held that the establishment of harbor lines does not remove the waters enclosed between them and the shore from the jurisdiction of the United States for such use as may be required for the aid of navigation, so long as the owner of the submerged land had not filled in "thereby practically removing his land from the body of the river and making it a part of the upland." He made no decision as to a case in which the fill had actually been made.*

The powers of the United States in the control and improvement of its navigable waters is paramount over all individual or corporate rights as riparian owner. In a decision rendered June 2, 1914, in the District Court of the Northern District of Illinois, in the case of *Franz Tempel* vs. *United States*, Judge Carpenter states:

> The title to the bed of the North Branch of the Chicago River in front of Tempel's property, whether in the state of Illinois or Franz Tempel, was subject to an easement in favor of the United States Government, and that any damages, caused by the dredging of said bed, to the bank along the shore line of said Tempel, are

*A later decision is found in *Garrison, Secretary of War, et al.* vs. *Greenleaf Johnson Lumber Co.*, Circuit Court of Appeals, Fourth Circuit, June 1, 1914, and is as follows:

"A harbor line established by the Secretary of War under authority conferred by Congress is subject to change by the same authority, and while a riparian owner may lawfully construct piers and docks to the established line, in doing so he takes the risk of such change if required for the improvement of navigation, which is not a matter for judicial inquiry, and the removal by the government of so much of his structures as extend beyond the new line is not a taking of his property for which he is entitled to compensation."

only incidental and consequential, for which the United States Government is not answerable in damages;

Said Tempel as a riparian owner along a navigable stream must himself bear the loss of any wearing away of his bank by erosion;

The North Branch of the Chicago River in front of Tempel's property being a navigable stream the dredging of the bottom of the river bed by the United States Government in 1899 did not constitute a taking within the meaning of the Fifth Amendment to the Constitution of the United States;

So, also, works for the benefit of navigation may be constructed on lands submerged by the navigable waters, and access to upland shore property may be denied by such works without the owner having any right of compensation against the United States.

This entire question is well covered by the following extract from the decision of the Supreme Court of the United States rendered in the October term, 1912, in the case of the *United States* vs. *The Chandler-Dunbar Water Power Co.* et al.

This title of the owner of fast land upon the shore of a navigable river to the bed of the river is at best a qualified one. It is a title which inheres in the ownership of the shore and, unless reserved or excluded by implication, passed with it as a shadow follows a substance, although capable of distinct ownership. It is subordinate to the public right of navigation, and however helpful in protecting the owner against the acts of third parties, is of no avail against the exercise of the great and absolute power of Congress over the improvement of navigable rivers. That power of use and control comes from the power to regulate commerce between the States and with foreign nations. It includes navigation, and subjects every navigable river to the control of Congress. All means having some positive relation to the end in view which are not forbidden by some other provision of the Constitution are admissible. If, on the judgment of Congress, the use of the bottom of the river is proper for the purpose of placing therein structures in aid of navigation, it is not thereby taking private property for a public use, for the owner's title was in its very nature subject to that use in the interest of public navigation. If its judgment be that structures placed in the river and upon such submerged land are an obstruction or hindrance to the proper use of the river for purposes of navigation, it may require their removal and forbid the use of the bed of the river by the owner in any way which in its judgment is injurious to the dominant right of navigation. So, also, it may permit the construction and maintenance of tunnels under or bridges over the river, and may require the removal of every such structure placed there with or without its license, the element of contract out of the way, which it shall require to be removed or altered as an obstruction to navigation.

The foregoing shows the nature and scope of the laws of the United States for the protection of the people's rights of navigation. Under these laws the Nation improves and maintains the waterways which have to be free and open for the use of all.

Just here arises the distinction between the functions of the national officials and those of the state or municipal officials charged with port duties. Those of the former relate to the general needs of commerce, those of the latter to the special transfer needs of the particular port. Both are of the highest importance, and there should be no clashing of interests. In its ultimate results any impairment of national interests must be detrimental to the local interests.

In addition to deep, safe and unobstructed channels of entrance and of communication, provided with ample and safe anchorages, the general requirements of a great commercial harbor are:

1. Accommodations for express passenger and package freight traffic. These should be of ample capacity, located conveniently to railroad terminals and to hotel and shopping centers, and connected therewith by highways unobstructed by railroad grade crossings and provided with facilities for rapid transit.
2. Piers for the direct interchange of traffic between water and railway carriers. These must also be of ample capacity and must adjoin large railroad yard areas, free from vehicular traffic. They should not be located in the heart of a city's water front.
3. Piers for freighters whose cargoes are distributed or received from many different sources. These should be located conveniently to the centers which they serve and should have ample means of access for motor and horse trucks with yard space for temporary storage of such vehicles. If possible they should have ample space for storage of incoming and outgoing freight with space for access to all parts for the instruments of transfer. Also, they should be interconnected by a railway with connections to trunk lines and possibly to bonded warehouses. The slips should be wide enough to permit of lighter service with other parts of the harbor.
4. Accommodations for pleasure craft, and for ferry service.
5. Ample water front for large manufacturing plants, which use water and rail carriers for raw materials and finished products. Such plants requiring large areas for proper development should not be located near the heart of a port.
6. Municipal piers open to all, some of which should be equipped with unloading plant or with coal handling plants for use of manufacturers whose works are located back from a water front.

It is evident that in a port which has grown simply by individual development and not according to a well thought-out plan, a wise distribution of the various classes of traffic is impossible, and an

uneconomical arrangement must exist with resulting financial loss. It is also evident that if the water front be divided into small privately owned parcels, the possibility of its proper development is practically nil.

Yet, it is only of recent years that effective steps have been taken toward systematic port organization and development on this side of the Atlantic. Municipal or state ownership and control of water fronts is essential to this end, and municipal or state authorities must control.

In the design of new ports or, what is more difficult, the adaptation of the older ports to the modern requirements for the economic movement of the world's products, there is required an exercise of ability of the highest grade.

The proper solution of the problems of transportation is one of the factors in the reduction of the cost of living. This increasing cost of living is a partial measure of the increase of comfort demanded for all classes, and to that extent is an index of advancing civilization. From this point of view it is not an inherently bad condition. However, to make its attendant hardships only temporary, and to supply this increasing demand for better living without an undue increase of the burdens of the people, it is necessary to seek all possible means of reducing cost. With each section of the world producing that for which it is economically best fitted, and with the means of transportation and distribution developed to the highest capacity, the burdens of to-day will be greatly reduced. The port authorities of the world can have a large influence in hastening or retarding the hoped for solution.

The establishment of the location of harbor lines has frequently formed a bone of contention between the national authorities and local interests. It is but natural that riparian owners should seek to increase the area and commercial facilities of their property by encroachments on the waterway. At times this can be permitted without detriment to general interests; at others it must be forbidden. Since the real interests of both the Nation and the locality are, in the broadest sense, the same, both the national and local authorities should be able to work in harmony in the establishment or re-arrangement of harbor lines. Experience has shown that where there is no carefully considered local control of a port, the establishment of harbor lines may be really detrimental. Cases can be cited where a short-sighted riparian owner has created solid fill just as far channelward as was permitted and has covered that fill

with his structures, but has later found that he had left himself insufficient pier area, with a result of a curtailment of the business capacity of his property. This has been followed by importunate requests for the further movement channelward of the pierhead line.

The remedy for this can be obtained by coöperation between the national and local authorities of the port. It is not always possible to foresee what areas of the port's shore line will be needed for manufacturing plants requiring limited berthing lengths and what areas will be required mainly for pier and slip purposes. Under these conditions, it is possible that one of two courses may be advisable. One, to establish simply a pierhead line which should be the limit of any construction of any kind channelward and then, as each property is to be developed, to have the plans for that development passed upon by the local authorities and construction authorized under a permit from the national authority. Under such a policy the application is made of record and the onus for any short-sighted division of the area in question between upland and pier and slip space is fixed on the property owner. In this connection it may be noted that experience in New York has shown that practically all of the circulation of water in the stream is between the opposed pierhead lines and that the effect of the space between each pierhead line and its corresponding bulkhead line is negligible in so far as the movement of the waters and the effect of such movement on the river bed and banks is to be considered. Where this is the case, open and solid fill piers have identical effects on the regimen of the waterway.

Another course would be not to establish formally any harbor lines but to lay down a theoretical pierhead line for the guidance of responsible officials, which should serve as a limit channelward for all structures for all shore developments and to have the right to make those developments up to that line, after having been properly and duly considered and recommended by local authorities, granted by a Federal permit for each individual case.

The operations of port protection and development, like any great public work, illustrate the truth of the Spanish proverb "You can't make an omelet without breaking eggs." Some interests are bound to suffer. Port officials, both Federal and local, must form judgment of the propriety of each project presented from the standpoint of the broad general interests of the Nation and of the locality and must be prepared to face pressure and criticism from those whose existing or prospective eggs have to be sacrificed.

Wooden and Combination Highway Bridges.

BY

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Like all of the other branches of engineering, the art and science of bridge construction has become so developed that it is no longer a single subject for specialization, but each of its many divisions may in itself well occupy the lifetime of a specialist. Not only do the features of design and methods of construction offer a wide field for selection, but the materials of construction, while comparatively few in number, present, in their variable combinations and overlapping fields of utility, studies that are almost unlimited in number. Of the structural materials suitable for use in bridges the most common are: steel, concrete (plain and reinforced), masonry and wood. The larger, more expensive and, scientifically, the most noteworthy bridges, include in their principal parts only the first three and, because of this fact, the last named material has almost ceased to be considered one of the important structural materials if current engineering literature may be considered an index of engineering thought.

It should not, however, be forgotten that wood still occupies an important part in many bridge structures, and it is a question whether or not greater familiarity with other structural materials does not in some instances cause their selection when wood could more economically (using the word in its broadest sense) be employed. Cases involving the following conditions should be particularly examined to prevent the error just suggested:

I. In many localities timber is cheap and the manufactured articles are expensive. Such conditions usually prevail in new and timbered countries where timber is close at hand and factories distant, and where transportation is expensive. Lumber may often be used and structures replaced at the ends of their periods of usefulness at less annual cost than the interest on the investment required for more permanent works.

II. The limitations of available funds often dictate the cheapest

construction regardless of economic loss in the long run. This is particularly common in the highways built by townships and counties in our western states.

III. The probable period of use of a structure may be short. Even though its period of use will probably be longer than the life of a single wooden structure, the economic advantage may be in favor of wood.

IV. Hasty construction to meet an unexpected emergency or to provide emergency repairs to damaged communications may be necessary.

The wide range of conditions covered by these four cases is evident. It will also be evident that cases I and IV may prevail on either roads or railroads, but cases II and III will rarely apply to the latter. In the above cases, number IV applies to most military bridges. The subject of hasty construction is covered in the "Engineer Field Manual" and will be dismissed from this discussion. The following paragraphs deal with the use of wood as an important structural material in highway bridges intended for permanent use during the life of the materials of which they are constructed. The subject matter is arranged under the three general heads of Design, Construction and Cost.

I. DESIGN.

Substructures.

Location. The most desirable type of substructure, and consequently the general type of a bridge, is fixed by the conditions at the bridge site. In the selection of the site, all of the details of bridge construction must be considered in a general way unless the site is fixed by other conditions than that of putting the most suitable bridge across a given stream. Usually the site is fixed within comparatively narrow limits, and the bridge engineer is concerned only with the comparisons of the various sites that are available within the limits beyond which the crossing may not be sought. A stream *can* be crossed anywhere, but the problem is to select the site that will allow the construction of a bridge to answer the desired purpose at the least ultimate cost.

Often the selection of the best site is easy, being evident to the most inexperienced eye; frequently it involves only a rough mental approximation of cost, but in many instances the most satisfactory site can be selected only by careful studies of the types required by the several available locations. In such cases, the knowledge of what can be done, how it is accomplished and how much it costs

in the particular locality is essential. No rules can be given, but the following are desirable features of the site, named in what is frequently their order of importance: Solid bottom, permanent banks, straight upstream reach, small width, low velocity current, single current or parallel currents, good approaches and shallow water. Few bridge sites afford all of these advantages, some offer few, and others none.

As noted in the preceding paragraph, the best type of bridge is fixed by the purpose to be served and the site selected, and where there is a choice of suitable sites the bridge required at each is more or less definitely determined in the processes of their comparison. In general, three conditions govern the design of the substructure: First, the supporting power of the foundation; second, the probable effect of the current, and third, the height of the grade crossing. Wood not being adapted to extreme spans, in the construction of wooden highway bridges, troublesome foundation pressures are rarely encountered. The current effects include scour, drift, and ice in addition to the dynamic force of the flowing water itself, which is usually insignificant. The height of grade crossing determines the proper spans by economic considerations when the most suitable spans are not determined by the preceding factors. Since the cost of piers of a given type and height increases with their number and the costs of superstructures increase with the length of the spans, for a given height and type of pier, there is a single span length of maximum economy. Generally, however, except in elaborate work, the advantages of standard construction avoid the necessity of hair-splitting studies. The type of wooden substructures suitable for various conditions will be outlined in the following paragraphs:

Trestle Bents. Because of the short span and consequent wide distribution of pressure, almost any soil is capable of supporting either framed or pile trestles, and they are at present more widely used than any other type of wooden substructure. They offer, however, slight resistance to current effects and, because of their obstruction to the channel, are particularly susceptible to damage by ice and drift, being liable to destruction either by the simple force of impact, or in unstable bottoms, by the increase of scour resulting from the lodgment of ice and drift against the bents.

Framed bents on mud sills can not be considered safe except in stable bottoms. Pile bents or bents on pile foundations are ordi-

narily safe against scour if the piles penetrate a solid clay stratum to a depth of 5 feet or more, and in softer bottoms if the penetration of the pile is at least equal to the maximum observed depth of water. Safety in the last case is contingent upon the prompt removal of drift from lodgment against the bridge. The dynamic effect of the current is not dangerous in current velocities ordinarily encountered, but drift in rapidly flowing streams may be dangerous and in streams having a distinct "break-up" or ice run, trestles can not be considered safe.

Simple trestles are not ordinarily applicable to sites having a high grade crossing. Heights up to 12 feet can usually be accepted with little suspicion, but in the interests of economy greater heights should be compared with longer span structures. The spacing of the bents may vary between the limits of 12 feet and 20 feet, 15 feet being a common spacing.

Framed Trestles are used where the soil near the surface is sufficiently compact to support the bents and where scouring effects are small. They are ordinarily constructed only where the water is very shallow. The posts should rest on a mud sill, usually a square or round timber with a width not less than that of the posts measured perpendicularly to the bent. The sill should be notched to receive the feet of the posts and the posts should be toe-nailed to it by one or more spikes in each side of the post. Framed bents may also rest upon pile foundations. Such use is limited to sites in which the bearing power of the soil is poor or difficulties in properly seating the mud sill exist, and where piles of sufficient length to form the entire bent are not available. The foundation is constructed as a pile bent, the cap of the bent taking the place of the mud sill. The bents are ordinarily four posts. The posts should be equally spaced, the tops of the outside posts falling under the guard rails of the bridge floor. In unusually wide bridges, additional posts can be used to decrease the size of the cap. Posts are designed as columns after any standard tables of strength.* A minimum diameter of 6 inches should usually be required. In

*For this, and the other computations mentioned in this paper, the tables in Kidder's Architects' and Builders' Pocketbook are particularly recommended. In the use of local timber due allowance must be made for the quality of the sticks secured. In Alaska, where the native spruce is injured by irregular growth due to the climatic conditions, the writer has usually employed a factor of 60 per cent in reducing the strengths given for spruce in Kidder's tables to obtain the strength of sound Alaska spruce. This factor is not too small.

bents under 6 feet high, the posts are vertical; in higher bents, the outside posts should usually be given a batter of 2 inches or 3 inches per foot. In bents higher than 6 feet, sway bracing of 3 by 10 planks, one on each side diagonally across the bent, bolted to the mud sill at one end, to the cap at the other, and spiked to the posts, should be employed. In lower bents, sway bracing should be used where current forces against the bent are to be expected.

The cap should be designed as a beam with its width that of the post measured perpendicularly to the bent. It may be given shallow notches to receive the tops of the posts, to which it should be drift bolted with one or more ½ inch or ⅝ inch drift bolts to each post.

Pile Trestles are used where foundation difficulties are met. These difficulties may be in the form of insufficient bearing strength of the surface soil, in the depth of the water, or in probable current effects. The number of piles may depend upon the bearing power developed; usually, however, not less than three piles to the bent are allowed, and that number is sufficient in most cases. In the determination of the safe load on a pile the *Engineering News* formula given below is recommended:

$$\text{Total safe load} = \frac{2wh}{s+1}$$

w=weight of hammer, pounds.
h=height of fall, feet.
s=penetration under last blow, inches.

It is of value only when the penetration under the last blows is approximately uniform. In extremely high bents, the strength of the piles as columns should be examined. Ordinarily, piles should not be under 12 inches in diameter at the butts and 6 inches at the tops. For convenience in driving, they should not exceed 18 inches in diameter at the butts. They may be driven with the bark on if the superstructure is to be made of logs or hewn timber unseasoned, or if driven in salt water. Otherwise they should be peeled, and if the superstructure is expensive and is to receive careful maintenance, painting, etc., they may be creosoted, depending upon the circumstances under which they are to serve and the cost of the process. Pile bents are capped and sway braced like frame bents. The bottom of the sway bracing is usually at a horizontal sash brace at the surface of the ground or at the low water line. The sash braces should be two 3 by 10 planks bolted to the piles, one on each side of the bent. High bents requiring storied construction or

longitudinal bracing are not common. When found economical, however, they are used, the longitudinal bracing and the sway bracing being computed from the estimated maximum wind loads, etc., instead of by the arbitrary adoption of the sizes given above.

Piers. When spans longer than 20 feet are desirable, the substructure of the bridge consists of abutments and piers. The supporting power of the foundation may require the adoption of long spans when the bottom of the stream presents variable soil conditions, so that suitable foundations can be secured only at certain points on the crossing. This condition is rare and the long spans are usually made desirable to avoid obstructing the channel, or because of the necessity of protecting each pier from scour, drift or ice, or because of the height of the grade crossing. The first and last conditions being questions of foundation pressures only, their solution depends merely upon special adaptations of the principle of trestle bents. Ordinarily, each pier is made up of two simple bents under the abutting ends of two consecutive spans. They are designed as ordinary framed or pile bents, taking into consideration the increased loads of the longer spans. They are braced to each other by a system of cross bracing if necessary—practically always if framed bents, and usually, if pile bents over 8 feet in height. The usual type of timber piers designed to withstand the effects of currents will be considered under the heads of *pile piers, crib piers,* and *pile and crib piers.*

Pile piers may be used when the spans have been increased to allow the passage of drift and may be considered useful in withstanding very weak ice flows. The length of span should be safely greater than the possible length of single pieces or booms of driftwood, in which case, properly designed and constructed piers can usually be depended upon. Pile piers under 10 feet in height can be expected to stand a run of ice; pieces not exceeding 10 cubic feet in size in currents up to about 4 miles an hour, or proportionately* smaller pieces in more rapid currents. For greater impact forces, their safety should be considered doubtful. The bearing power of piles carrying vertical loads can be determined as previously noted. If piles are expected to withstand forces having large longitudinal components they can be examined as cantilevers where the stresses are calculable. Piles to withstand the impact forces

*In this, and similar proportions mentioned later, the mass of the piece is inversely proportioned to the square of the velocity of the current.

of ice and drift should penetrate a firm stratum—clay or gravel—not less than 6 feet nor less than the distance from the highest point of impact to the surface of the ground. Pile piers can not be depended upon where thick ice can freeze to the piles and be lifted by rising water. The general remarks previously made in regard to peeling, creosoting, etc., apply to pile piers.

A pile pier of the type under consideration consists of two parts: the supporting pier, or main pier, and the nose, or guard pier. The main pier is composed of two pile bents separated by 4 feet to 8 feet. The number of piles, the sway bracing, and the longitudinal cross bracing are dependent upon the penetration, the bearing power of the piles, the length of the spans and the height of the piers, according to the principles previously indicated. Cross bracing is important if the piles of the guard pier are braced to the piles of the supporting pier. The nose or guard pier may consist of a single dolphin or group of three or more piles driven as closely together as possible and with their tops bound together by a cable or metal strap. This should resist ice and drift under conditions that make the stability of pile piers of some little doubt, but can not be expected to resist ice or drift in swift currents. A better form of nose consists of several piles driven in a row upstream from, and opposite, the center line of the supporting pier. The upstream pile should be cut off 2 or 3 feet below the probable height of the water line at the time of drift or ice flow. The other piles should be cut off at increasing heights toward the pier and all should be cut at such an angle that a nose log can be drift-bolted to them. The angle of the nose log with the horizontal may vary from 30° to 40°, decreasing as the probable impact forces increase. The nose log should be about 12 inches in diameter, should be supported at not greater than 4-foot intervals, and should be drift-bolted to each pile with two ½ inch or ⅝ inch drift-bolts, one on each side of the center line of the nose log. A light metal strap or a light railroad rail should be spiked to a round nose log if ice is to be resisted. If the nose log is square, the rail, or two small angles, one at each upper edge, may be used. This form of nose is capable of standing heavy impact forces if the current is parallel to the center line of the pier. It can not, however, be expected to resist an ice jam.

A more elaborate and stronger type of pier is shown in Fig. 1. It is about the best type that can be built of piling, but a pier stronger than that shown in the figure can be built by decreasing the spacing of the piles, adding a waling piece at the top of the piles in the wings, and bracing the waling pieces to the sides of the nose

and the supporting pier. The waling pieces should have a minimum diameter of 6 inches and should be bolted with one ⅝-inch bolt to each pile. A pier of this kind can be expected to stand moderate ice flows, even light jams, and almost any drift other than large booms of logs.

Many of the functions of *crib piers* overlap those of pile piers to a considerable extent, most of the functions of pile piers being equally well performed by cribs. In such cases the particular type to be employed depends upon the facilities for erection. Crib piers, however, have certain special adaptabilities and limitations. They are more suitable than pile piers in locations subject to variations in the direction of currents at varying water stages, to locations where hard strata are at insufficient depths to give lateral support to piles, and to locations subject to ice jams. They are not so suitable for unstable bottoms (though they may be used with mattress protection), for sites in which confinement of the channel is objectionable, and for sites requiring piers of great height. Well constructed crib piers over 8 feet in height should stand a considerable amount of impact from drift and ice. They are safe under almost all ordinary conditions of drift and should stand an ice run with pieces about 3 cubic yards in a current of 4 miles an hour and proportionately smaller pieces in swifter currents.

Cribs may be built of logs or squared timbers. The sticks used should have a minimum diameter of 8 inches. If logs are used, they may be peeled, but in most cases the advantage of peeling does not warrant the expense. The main body of a crib will usually be rectangular, 16 feet to 24 feet in length and 8 feet to 12 feet in width, inside dimensions. The exact size of the pier depends upon the weights to be supported, the impact forces to be resisted, and the height required. It is ordinarily divided into compartments by cross ties, separated by about 4 feet horizontally and that distance or less vertically. If the crib is to sink into a soft foundation, the first row of ties should be far enough above the bottom of the crib to permit the required sinking and a floor should be put in on these ties. Otherwise, flooring is not required unless the crib is to be sunk in water deep enough to float the crib. In this case a portion of the crib may be floored.

The logs or timbers may be halved together at the corners, making a tight crib, or simply laid alternately, a long log and then a cross log, making a crib with openings the size of the timbers. Usually, with round logs, both logs should be flattened to a bearing surface about 4 inches wide on each side, and with squared timbers they

should be laid without notching. Each log or timber is bolted to the log below with one $\frac{5}{8}$-inch drift-bolt long enough to reach through the first log below and extend about 2 inches into the second log. These drift-bolts may be shorter above and below the points at which impact forces are to be expected, except that in piers over 10 feet high the lower bolts should not be shortened. In addition to the rectangular body, piers subject to the attack of drift and ice should be given a wedge-shaped nose upstream, and in unstable bottoms with rapid currents, a short wedge-shaped tail. The body of the pier should be built like a rectangular pier, but with the long logs

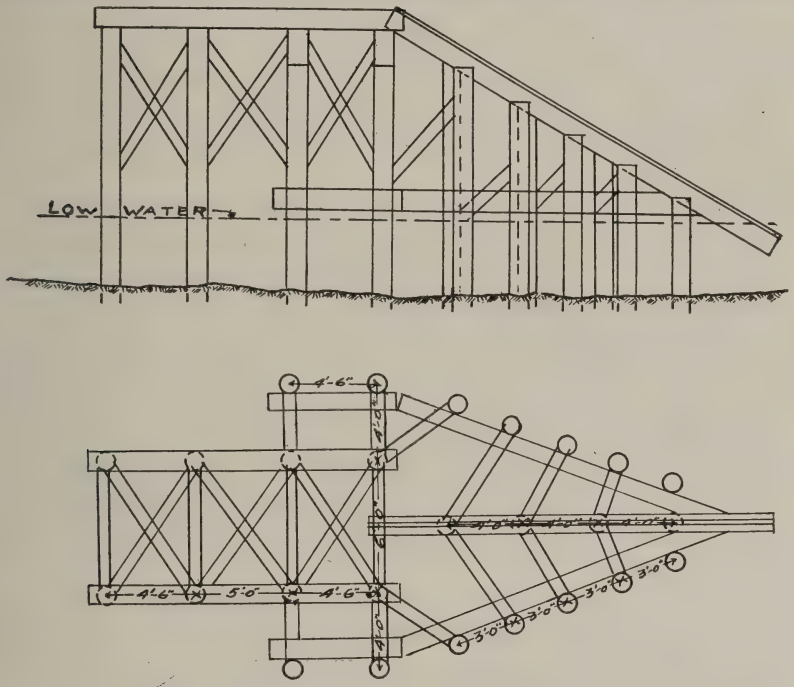


Fig. 1. Plan and Elevation of Typical Pile Pier.

extending beyond the cross logs a sufficient distance to allow them to be framed and drift-bolted to the wedge logs. The upstream nose should generally have a vertex angle at the line of greatest impact of about 60 degrees. The wedge logs should be framed at the vertex as described for the corner of a rectangular pier, the ends extending sufficiently to allow a bearing of not less than 6 inches for the nose log. The vertex angle of the wedge increases from bottom to top to give the nose log a slope. The remarks regarding the size, slope, metal protection and bolting of the nose of a pile pier, apply to the nose of a crib pier. The tail wedge is usually not necessary. When

required to prevent the formation of a suction eddy, it is usually made with a vertex angle of 90 degrees. It is framed like the nose, but without a nose log and with a constant angle at the vertex, making the tail line vertical. The tail wedge should be carried to the high-water line.

Cribs are filled with stone, at least to the highest line of probable impact. The size of the stone is unimportant, except that it must be larger than the openings between crib logs and is usually large enough to be handled one stone at a time and not larger than one or two-man stone. The stone should be placed well against the walls, smaller pieces being wedged into spaces between larger stones and the crib logs. The spans of the superstructure rest on caps, one for each span. The caps usually rest on stringers which are supported by the crib walls, from which the caps are set back 2 feet or more, frequently being placed one-quarter of the distance from one wall to the other.

Pile and crib piers are used where strength greater than that obtainable with simple cribs is desired. They are particularly useful in soft bottoms where cribs can not be expected to stand without settlement from time to time, unless sunk to impracticable depths. When the length of the crib is small, the addition of the piles greatly increases its resistance to impact; as the height increases the value of the piles becomes less. In cribs that may reasonably be suspected of liability to even small settlement, the supporting piles should be added if the superstructures are expensive. A well constructed pier of this type will resist a heavy ice run, and by increasing the dimensions can probably be expected to withstand any but the most severe ice jams.*

The simplest form of pile and crib pier is composed of a crib constructed according to the principles already given, but with the trusses supported on pile bents extending through the cribs and driven into the underlying bottom. The caps of the piers take the place of and occupy the positions recommended for the crib caps, the crib stringers being omitted. A complete pile and crib pier is constructed like the crib pier described above, but with an additional pile at each corner of the rectangular portion of the pier and with piles along and against the walls of the rectangle and the nose. All

*A pile and crib pier built in the Gulkana River, at Gulkana, Alaska, has successfully withstood two spring ice runs. One of these is reported by local observers as having been terrific, but no accurate information regarding the velocity of flow and the size of the bergs has been obtained. The pier was constructed and allowed to stand one spring ice run before the superstructure was put upon it.

of the piles are inside of the cribs and should be about 4 feet apart. The logs are then drift-bolted to the piles with ⅝-inch drift-bolts. This construction adds greatly to the strength of the pier, being particularly a protective measure against damage to the pier and the supporting bents caused by excessive settlement of the pier itself. Piers of this type should not be floored, and when the filling settles the crib should be filled as soon as possible.

Abutments. The approaches to bridges of wood will usually be pile or framed bents, if of considerable length, and in any case will usually terminate in a single mud sill for the support of the floor stringers. When built-up abutments are necessary, the best form of wooden construction is usually a vertical river wall parallel to the

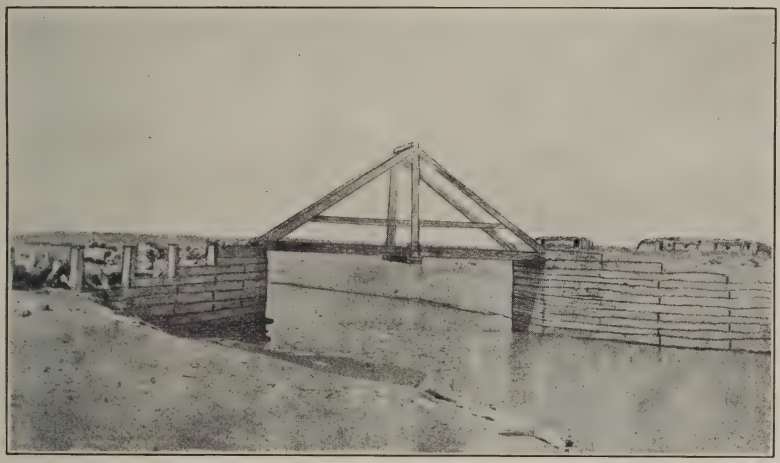


Fig. 2. Hastings Creek Bridge, Seward Peninsula, Alaska. Thirty-foot Span of Douglas Fir.

bank of the river and with vertical wing walls. The river wall and wing walls are constructed like a side wall and the corresponding wedge walls of a crib pier. The wing walls, when oblique, usually make an angle of 45 degrees or less with the river wall. The ties of the crib construction are replaced by headers extending into the bank or fill behind the abutments, according to the rules for spacing given for the ties. Frequently the wing walls will be perpendicular to the river wall, and when so placed ties can be omitted. Another type of abutment constructed of piles and sheathing is shown in Fig. 2.

If scour beneath the abutment is expected, the abutment may be supported by piles driven behind and against the wall. The logs of

the wall should be drift-bolted to the piles. In this case, the filling next to the wall should be of stone, and if it sinks it should be replaced as soon as practicable. Abutments built as indicated can be considered safe under any conditions of ice and drift that make pile and crib piers permissible. Their greatest danger is from the effects of bank erosion, which must be cared for when necessary by mattress or other form of bank protection.

Superstructures.

Loads and strengths. The loads on a highway bridge are classified as dead loads, snow loads, live loads, and wind loads. The *dead load* consists of the weight of the bridge itself. For purposes of design the weight should be assumed, and after the design is made the actual weight should be computed, and if materially differing from that assumed, the design should be changed accordingly. The floor system should be designed after the general type of truss has been selected and its actual weight computed. The probable weights of the trusses may then be computed from the formula:

$w = 1/6\ (b+45)\ L.$

w=weight of trusses, bracing, etc. (except floor system) per linear foot of span, pounds.
b=width of clear roadway, feet.
L=span, feet.

In computing the actual weights of various parts of the structure, the tables in any standard handbook may be used. The weight assigned to wooden members should be the weight of wood wet, as after a continued rain. This weight varies little with various classes of wood and may be assumed at 4.5 pounds per foot board measure. Iron and steel weigh 490 pounds per cubic foot.

The *snow load* varies according to climatic conditions. It may be zero, and in rare instances 60 pounds per square foot. Light snow weighs from 5 to 12 pounds, and packed snow weighs 15 to 45 pounds per cubic foot. Where a long series of weather observations is obtainable, the writer recommends that the maximum observed annual snowfall in inches be multiplied by 0.1 to obtain the proper snow load in pounds per square foot for the design of the bridges that are not to be regularly cleaned of snow. For bridges regularly cleaned, the maximum recorded snowfall in inches for a single storm should be multiplied by 0.9 to obtain a proper figure for the snow load in pounds per square foot.

The *live loads* on bridges for general traffic are largely matters of assumption. Table I gives the commonly accepted standard

I. Table of Live Loads for Highway Bridges.

Traffic.	Floor and its supports. Equally distributed on four wheels, 5-foot axles at 10-foot centers.	Trusses.			
		100-foot spans.*		200-foot spans.*	
		Per linear foot each car track width occupied 12 feet.	Per square foot remaining space	Per linear foot each car track.	Per square foot remaining space
		Pounds.	*Pounds.*	*Pounds.*	*Pounds.*
City	24 tons	1,800	100	1,800	80
Suburban	24 tons on car tracks; 12 tons any part of roadway	1,800	80	1,200	60
Heavy country	18 tons on car tracks; 12 tons any part of roadway	1,200	80	1,000	60
Ordinary country	6 tons any part of roadway or 80 per square foot of floor surface	----------	75	----------	55

Where a steam roller is likely to be used on the bridge, it should be provided for. The heaviest type gives a loading as indicated in sketch (Fig. 13, Plate I).

*Intermediate spans by interpolation.

loadings for highway bridges. The column headed "Floors and Supports" gives the loadings assumed in the design of floor stringers, roadway beams and short vertical hangers, as in king-post trusses or in subdivided panels in long trusses. The columns headed "Trusses" give the loadings assumed for the truss computations. These loadings are recommended for use when the traffic can not be more closely classified than in those tables. In many instances the largest load that can reasonably be expected to come upon the bridge during its life can be more accurately prophesied, and it should be used instead of the loading of the general table. The necessity of providing for the large uniform load does not appear to be usually necessary in outlying districts. If designed, however,

Fig. 3. Stewart Creek Bridge, Valdez-Fairbanks Road, Alaska, showing snow-load, January, 1914. Two 60-foot spans of Alaska Spruce.

for less than these standard loads, the bridge should be protected against overloading by suitable signs.

The *impact stresses* of the live load are indeterminate. They are small when the speed of the traffic using the bridge is low and when the bridge floor is smooth. The writer believes that an impact allowance of 25 per cent of the stresses produced by the live load statically applied is sufficient. In the design of bridges for the Alaska Road Commission, however, where the floors are frequently of poles, this allowance has been increased to 40 per cent.

The top and bottom lateral bracing should be designed to carry a *wind load* of 30 pounds per square foot of the surface exposed in

side elevation. This load should be treated as a statically applied load, equally divided between upper and lower panel points. In addition, a wind load of 150 pounds per linear foot for the distance equal to the longest load expected to use the bridge should be treated as a possible live load applied only at the lower panel points. In cases to which the standard loadings of Table I apply, this live wind load covers the entire bridge. The stability of the bridge against being overturned by the wind when unloaded should be examined.

The *strengths of members* may be computed from any standard strength tables. Kidder's tables, previously mentioned, give safe loads suitable for direct application in bridge design when the usual kinds of timber found in the United States are to be used. When these tables are not available, the strength of members should be computed from ultimate strengths, using the following factors of safety: Tension truss members: wood, 8; iron or steel, 4. Compression truss members: wood, 5; iron or steel, 3. Beams (max. tensile fiber stress): wood, 6; iron or steel, 3. Bearing, shearing and splitting strength: wood, 5; iron or steel, 3.

Floors. The floors of highway bridges will usually be entirely of wood if the relative cost of that material warrants its use in the important members of the superstructure. Other types of floors will be omitted from this discussion.

The floor system consists of a floor or deck resting on longitudinal stringers which are carried by roadway bearers or floor beams supported at the panel points. Wooden decking may be laid in one or two layers. If the traffic using the bridge is dense the latter is advisable, to permit the frequent replacement of the roadway surface without closing the bridge to traffic. The lower layer should usually consist of 3-inch plank laid transversely and the upper layer of 2 inch or 3 inch plank laid diagonally. Single decking should be 3 inch or 4 inch plank laid transversely. The width of the roadway is determined by conditions depending upon the traffic to be served. The panel lengths are then fixed. This is usually a somewhat arbitrary choice depending upon the length of the span, the structural sizes conveniently available, and the weight of the expected traffic. If an exact span has been fixed by other conditions the panel length must be a factor of the total span length, so that a whole number of equal panels may be employed (it is, of course, not theoretically necessary that the panel lengths be equal, but in practice they should be). The panel lengths selected will ordinarily

vary from 12 to 20 feet, lengths of 12 to 15 feet being common for light bridges and 15 to 20 feet for heavy bridges.*

The minimum number of stringers is fixed by the width of the roadway. Stringers should not be separated by more than 3 feet, and the outside ends of the decking should rest on stringers. The sizes of stringers required for this spacing is determined by computing their strength as beams to support the deck, snow and most adverse live load for each stringer. The width of the stringer should be at least 3 inches and not less than one-quarter its depth. The number of stringers may then be varied to give an economical adaptation of commercial sizes.† The stringers of adjacent panels may lap at the roadway bearers, each stringer extending over the entire width of the bearer, or they may be butted together, each stringer covering half the width of the bearer. When so placed they should be secured by one and preferably two fish-plates spiked to the stringers and notched as indicated in Fig. 4, Plate I. In either case, the stringer should be toe-nailed to the beam.

Having fixed the number and spacing of the stringers, the minimum width of the roadway bearer can be determined by the minimum allowable bearing area of the stringer under the maximum load and the width of the stringer. The required depth of the roadway bearer is then computed to safely support all stringers under the dead, snow and live load with the concentrated portion of the live load supported on the two central stringers, and a suitable commercial size is selected.

A guard rail, frequently a 6 inch by 6 inch piece, should be placed at each edge of the decking and a hand rail, about 4 inches by 4 inches, supported 40 inches above it. The guard rail may be bolted to the decking or fastened through the deck to the outside stringer by means of small wood screws to facilitate removal. The hand rail in through trusses is usually spiked to the truss members. In trestles and deck trusses, it should be supported by posts, braced

*The reasons for this apparently illogical practice arise from the considerations that the stringer spacing should not generally exceed 3 feet to avoid extremely heavy decking and that for lateral stiffness and stability the thickness of the stringer should not be less than one-fourth its depth. Also, in long-panel trusses under light loads, the size of compression members may be increased by the requirements of stiffness.

†This and similar references to commercial sizes do not apply, of course, when round timbers are employed, but the adjustment of sizes may be necessary to reduce the required diameters to available sticks.

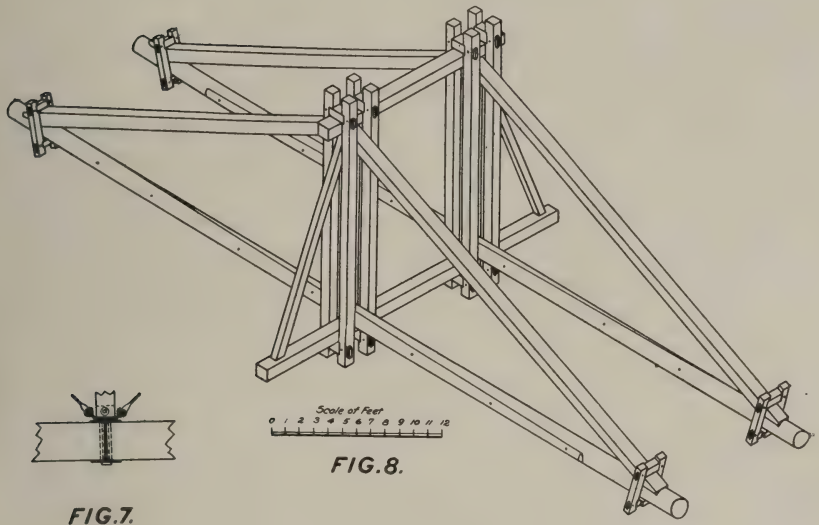


FIG.8.

FIG.7.

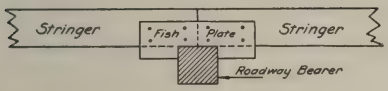


FIG.4.

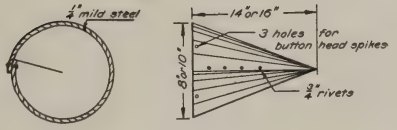


FIG.9.

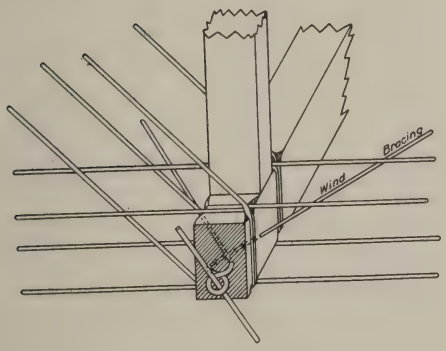


FIG.12.

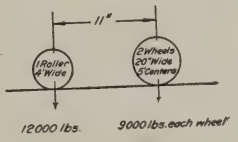


FIG.13.

PLATE I.

by outside braces resting on projecting planks of the decking, long planks being laid for this purpose at suitable intervals.

Trusses. Spans 20 feet, and in rare instances 25 feet in length, may be accommodated by the floor systems alone. For greater lengths, however, trusses or suspension spans must be constructed to support the roadway bearers at the proper intervals. Permanent suspension spans can be constructed only with iron or steel in the principal members, and wood in such construction is confined to the towers, floor system, lateral trusses and stiffening trusses, wherein its uses are not essentially different from those in supporting trusses. Suspension bridges fall beyond the scope of this paper.

Bridges in which wood plays an important part are classified as wooden or combination bridges, depending upon the members in which wood appears. Wooden bridges include all bridges in which both chords are of wood. Combination bridges are those in which the tension members are of iron or steel and the compression members are of wood. Owing to the difficulty of making economical tension joints in wood, and to the greater facility of erection of bridges with metal tension web members, the latter are almost always used even in wooden bridges. So that, generally speaking, a wooden bridge is one with wooden chords and wooden compression members. The combination bridge is a development of wooden construction in which the necessity of tension joints in wood is obviated, in which it is practicable to construct true pin-connected structures, and yet in which the economy of using wood under the stresses (compressive) that it is best fitted to resist, is obtained. The selection of a type to be used in a particular case should be made the subject of examination according to the economical principles considered in the first part of this paper, unless the truss spans are to exceed 150 feet in length. For spans greater than 150 feet in length, the use of combination trusses is recommended in all but the rarest cases. Either wooden or combination spans may be built in any of the usual types of trusses, but the Howe and Pratt types are the most common. Complicated trusses with subdivided panels, and cantilever trusses are rare since, as previously noted, for the heaviest uses and longest spans, wooden construction does not present the most desirable features. Of the two common types of trusses, the Howe type, because of its shorter web members in tension and their vertical position, which simplifies the framing, is pre-eminently the type for wooden trusses. The Pratt type, for converse reasons, is the better type of combination truss. The king post and

queen post trusses are limiting cases of these types. The spans may be either through or deck spans without affecting the essential features of design. The height of the truss is chiefly a question of economy of materials, subject to the limitation that through trusses should give a free clearance or head room of not less than 14 feet. Pony trusses, because of the sizes of the members and the difficulty of giving suitable wind bracing, are generally satisfactory only for use up to 50 feet in length. For this and greater spans up to 150 feet, heights varying from 14 feet to 20 feet, measured between the center lines of the chords, may be roughly selected by interpolation. The exact height chosen should allow the diagonal members to be cut from commercial sizes with minimum waste.

Within the limits of this paper a complete discussion of the advantages and disadvantages of the many details of chords, joints, angle-blocks, etc., is impracticable. The following brief notations, however, cite details that the writer considers the best for their several purposes:

Wooden Howe truss details. The details of a very light 75-foot truss designed for use in certain parts of Alaska is shown in Fig. 5. This truss is designed for a snow load of 30 pounds per square foot and for a single concentrated live load of six horses drawing bob-sled, each horse weighing 1,600 pounds and the total gross weight of the sled being 12,000 pounds. The floor is designed for this load as actually distributed with the sled on any part of the roadway. For the design of the trusses, the load was conventionalized to 11,000 pounds on each of any two adjacent roadway bearers. The materials used were Douglas fir for all wooden truss members, and mild steel for tension members. In the bridges actually built to date the floors have been constructed of native spruce, round stringers and pole decking being used.

It will be noted that the lower chord is built up of planks bolted together. In lieu of this form of construction, a better form of chord is one composed of thicker planks separated by fillers, pieces being jointed by a filler designed as a tabled fish-plate. In the particular case in question, the life of the Douglas fir in the trusses being considerably longer than the native spruce substructures, the additional expense of the better form of chord was not thought warranted. The lengths of the planks are chosen so that the notched angle-blocks fall between joints and do not decrease the net tensile area. For convenience in framing, the top and bottom chords have the same width and the section in each case remains uniform throughout the length

of the chord. This practice is general except for extreme spans. In the lower chord advantage of the excessive cross section in the end panel is taken in designing the end post notch.

The joints of the upper chord are arranged to fall at panel posts. The inset angle-block has no theoretical effect upon the strength of the upper chord. Practically, the unavoidable inaccuracies of framing produce slightly eccentric loading in the chord. No special allowance is made for this in the computations.

The verticals are mild steel rods upset to allow for the thread and properly washered with plate washers. They are carried outside the chords by the top wind struts and the roadway bearers. The diagonals are given such directions that the center lines of all intersecting members pass through a common point.

The horizontal trusses of the wind bracing are, for convenience in framing, made to receive their loads by the friction between the top struts and roadway bearers and the chords assisted by the vertical rods acting as short cantilevers. In long trusses the advantages of framing would be more than offset by the eccentricity of the loadings resulting. The side wind braces at the ends of the upper chords carry the wind loads to the bottom truss. The size of the end verticals is increased to cover the additional stresses they receive as members of this cross frame. The stresses transmitted by the side braces are a part of the loading of the bottom wind truss. The ends of the bottom truss are in the pier caps which are secured against horizontal motion by their fastenings to the piers. In longer trusses the caps should be notched into the bottom chord, or the rods of the wind trusses should be carried through blocks secured to the chord.

Combination Pratt truss details. The sketches in Fig. 6 illustrate satisfactory details of a combination truss. The sketches are to scale, but the sizes of the members as drawn are assumed, the sketches being merely for illustrative purposes.

The lower chord is composed of flat eyebars, each member consisting of two bars to permit proper arrangement on the pins. The end post foots in a casting which carries the end post pin. This pin carries the adjacent lower chord members and the diagonal members of the lower wind bracing. The roadway bearers are supported by one stirrup at each end, hung to the pin between the chord eyebars.

The upper chord is jointed at each panel. The joint is made by seating the chord member into a casting carrying the pin. All the

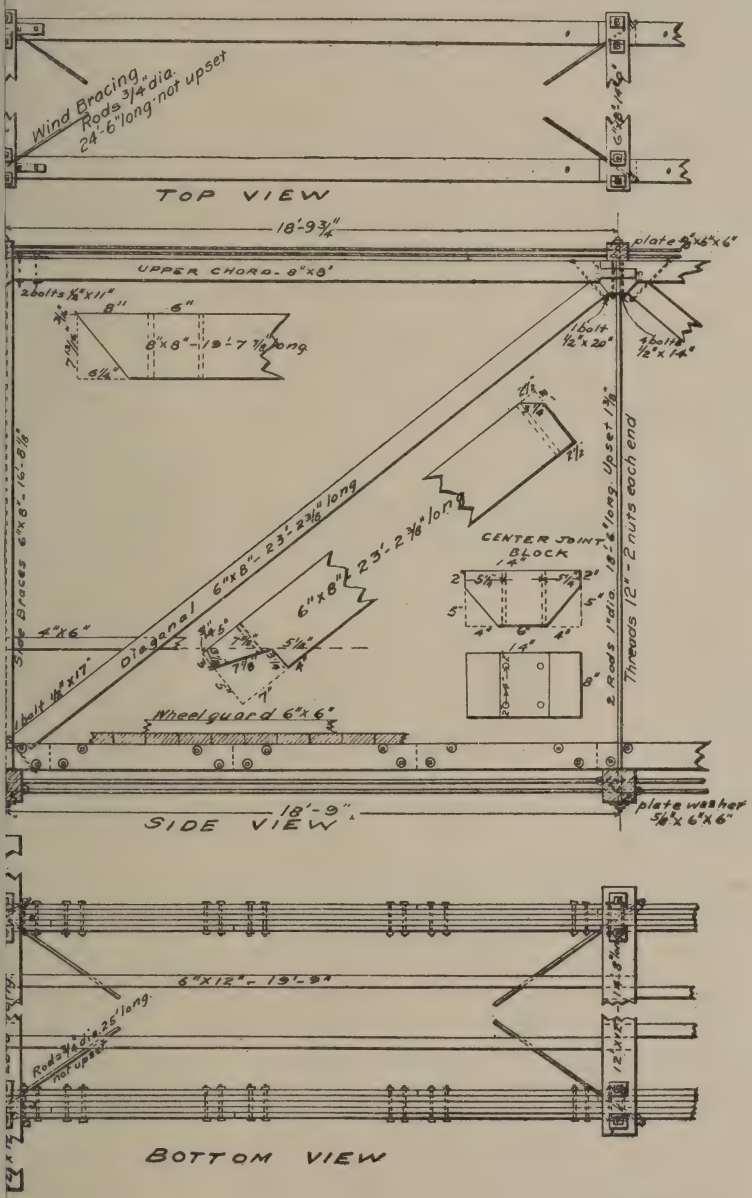

Wind Bracing
Rods 3/4" dia.
24'-6" long - not upset
TOP VIEW
18'-9 1/4"
UPPER CHORD - 8"x8"
plate 5/8"x6"x6"
2 bolts 1/2"x11"
8"
6"
8"x8" - 13'-7 7/8" long
1 bolt 1/2"x20"
1 bolt 1/2"x14"
6"x8" - 23'-2 3/8" long
Diagonal 6"x8" - 23'-2 3/8" long
CENTER JOINT BLOCK
Side Braces 6"x8" - 16'-8 1/2"
4"x6"
1 bolt 1/2"x17"
Wheelguard 6"x6"
2 Rods 1" dia. 18'-6" long. Upset 1 3/8" Threads 12" - 2 nuts each end
18'-9"
SIDE VIEW
plate washer 5/8"x6"x6"
6"x12" - 19'-9"
Rods 3/4" dia. 25' long. not upset
BOTTOM VIEW

FIG.5.

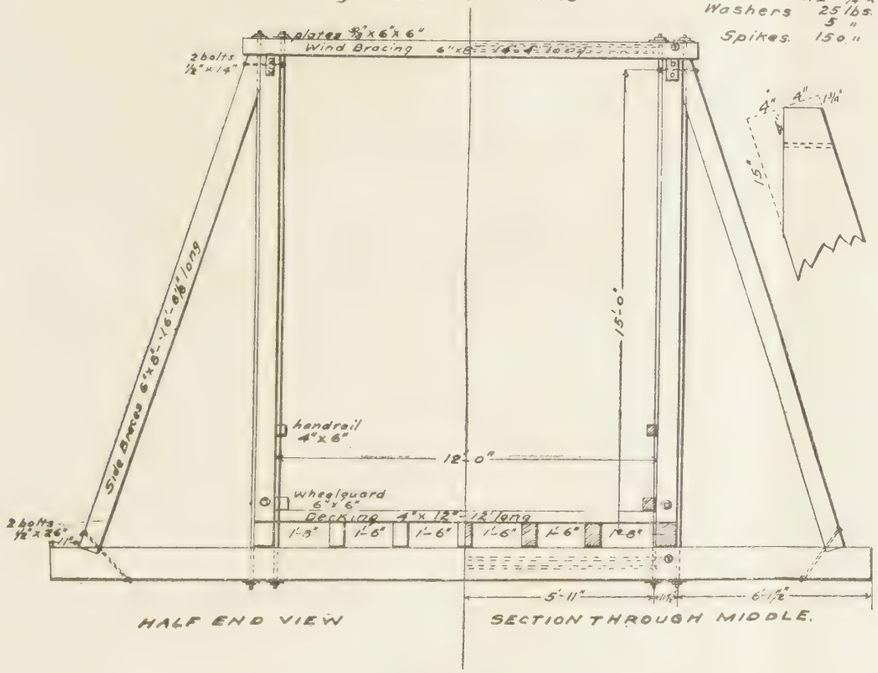
BILL OF MATERIAL FOR TWO TRUSSES.

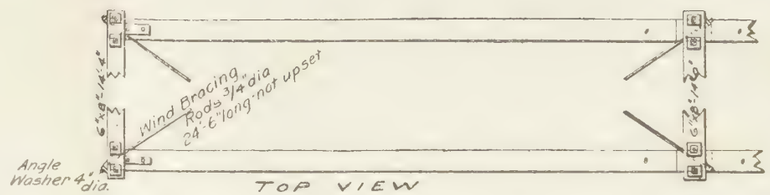
WOOD

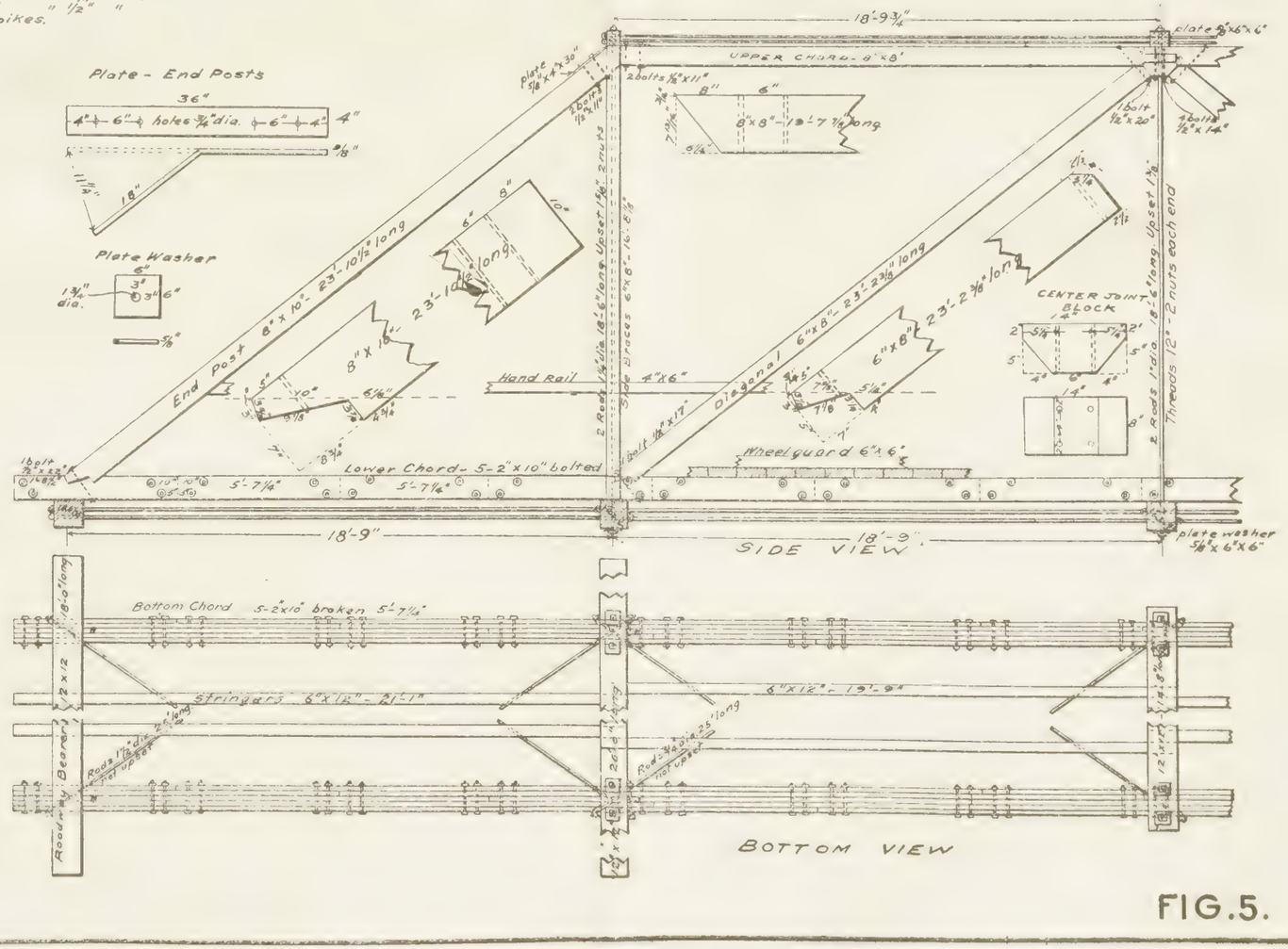
MEMBER		DIMENSIONS.
Upper Chord	4 pcs	8"x8"-19'-7⅞" long
Lower Chord	2 "	2"x10"-28'-0" "
End Post	4 "	8"x10"-23'-10½" "
Road Way Bearers	2 "	12"x12"-18'-0" "
	2 "	12"x12"-26'-0" "
	1 "	12"x12"-19'-8" "
Stringers	10 "	6"x12"-21'-1" "
	10 "	6"x12"-19'-9" "
Diagonals	4 "	6"x8"-23'-2⅜" "
Side Braces	4 "	6"x8"-16'-8⅛" "
Wind Braces	3 "	6"x8"-14'-4" "
Blocks	2 "	7"x8"x12" "
Decking	78 "	4"x12"-12'-0" "
Handrails	160 linear ft.	4"x6"
Wheelguards	160 "	6"x6"

IRON

MEMBER	DIMENSIONS
Vertical Rods	8-1¼" dia-18'-6" long Upset 1⅝"-12" threads each end 2 eq. nuts each end.
	4-1" " -18'-6" " " 1⅛" " " " " " " " " "
Wind Braces	4-1½" " -25'-0" " not Upset 6" " " " 1" " " "
	4-¾" " -25'-0" " " " " " " " " " " " " "
	4-¾" " -24'-6" " " " " " " " " " " " " "
Angle Washers	16-4" dia for ¾" rods.
	8-5" " " 1½" "
Plate "	24-⅝" x 6" x6"- 1hole 1¾" dia
Angle Plates	4-⅝" x 4" x 36"
Bolts	8-½" x 20"
	24-½" X 14"
	4-½" x 22"
	8-⅝" X 26"
	8-½" X 11"
	112-¾" x 12"
Washers	25 lbs. cut washers for ¾" bolts.
	3 " " " " 1½" "
Spikes.	150 " 6" wire spikes.







75 FOOT TRUSS

DOUGLAS FIR

ALASKA ROAD COMMISSION

Scale of Feet

1 0 1 2 3 4 5 6 7 8 9 10

FIG.5.

diagonals and the vertical of the first panel are loop-welded rods, two being used for each member. These rods are usually turnbuckle rods except in long spans. The vertical web members transmit their stresses to the lower pins by means of shoes made of plates riveted to the legs of a channel. They extend beyond the back of the channel

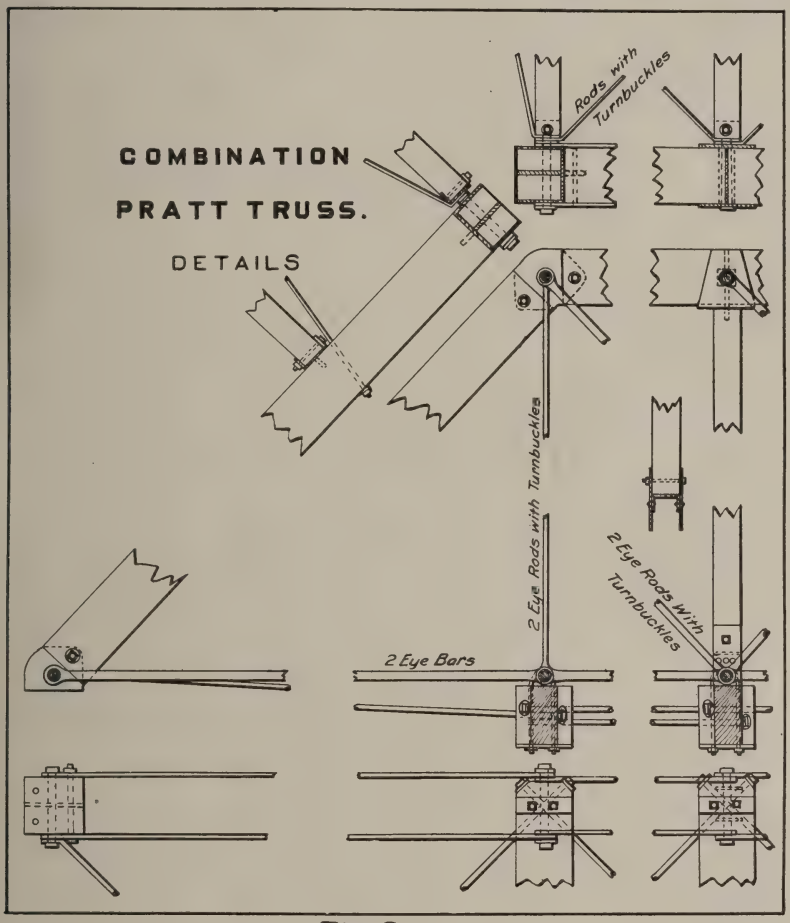


Fig. 6.

to be bolted to the vertical and beyond the legs of the channel. Beyond the legs of the channel they are drilled to receive the pin. The upper end of the vertical may be doweled or bolted to the upper chord casting.

The upper wind trusses are composed of wooden struts ending in steel angles. The vertical leg of the angle is drilled and threaded

to receive one end of the pin. The horizontal leg acts as a shelf to support the weight of the strut. A bolt through this leg and the strut holds the strut in place. The diagonals are loop-welded bars bent to the proper angle and carried by the pins. The rods should be turnbuckle rods. A more satisfactory device in long spans consists of clevise rods cotter-pinned to a wing-plate packed on the pin. (See Fig. 7, Plate I.)

The loads of the upper truss may be carried by portal bracing and the end posts, or by side braces as indicated for the wooden truss, Fig. 5. The bottom truss is similar to that in the wooden truss except that the end diagonals are looped at one end to receive the end post pin and are bent to the proper angle, or are clevised and carried to the pin by wing-plates.

An adaptation of combination construction that has been successfully employed, consists of the use of steel cable for the tension members of Pratt trusses. The roadway bearers and wind struts themselves take the place of the pins. The verticals are square end struts between the top and bottom chords to which they are spiked during construction. Each tension member is formed by a length of cable doubled around and resting in a groove of a roadway bearer or a wind strut. The top wind struts are knee-braced to the verticals so that each panel is a complete rectangular frame and can be so erected without the use of false work above the lower chord line. Eight 90-foot trusses of this type were constructed at Fort Riley, Kans., after a design by and under the direction of Maj. (now Lieut. Col.) Thos. H. Rees, Corps of Engineers. This type of construction appears to be of particular value for rapid construction by comparatively unskilled workmen and with small plant and meager equipment.

II. CONSTRUCTION.

Only general features of construction usually applicable to all cases will be mentioned under the heads of Organization and Equipment and Erection. Construction features, while treated separately in this paper, are important considerations of the design as they usually affect and sometimes govern the type of work to be undertaken. This is particularly true in small or isolated projects where the desirability of using a single crew on various classes of work and the limitations of meager plant are particularly urgent. In many projects under the Alaska Road Commission, the difficulty of transportation has limited very narrowly the plant that could

be landed at the bridge site. Such conditions caused the construction of the trusses shown in Fig. 8, Plate I, several of which were built by Foreman Lars Holland, for the Quartermaster Department, U. S. A., in 1900, in locations where the cost of transportation on all supplies and equipment exceeded 20 cents per pound. Three bridges, each with two truss spans, were built with no equipment except axes, saws, a 2-inch ship augur, chisels and pocket knives. Only native timber was available and no metal of any kind was used in any part of the bridges, and the bridges were built entirely without plans. These bridges were in use for periods varying from ten to thirteen years, and one at the latter age successfully carried a load of six horses drawing nine tons on two sleds in addition to a light snowload.

Organization and equipment. A satisfactory crew for the construction of bridges involving one or more long span trusses of the types under consideration should have about the following personnel:

Bridge Crew—
1 foreman,
1 head carpenter,
2 carpenters,
1 blacksmith,
1 engine runner,
1 teamster,
10 laborers,
1 cook,
1 cook's helper.

The number of teamsters actually engaged will depend upon the amount of material to be hauled. One teamster and his team will almost always be necessary at the bridge, in addition to the transportation required to land materials and supplies at the bridge site. The blacksmith may not be necessary, if access to shops is convenient. The engine runner is, of course, required only if a pile driving, hoisting or logging engine is to be used. Three of the laborers should be qualified carpenter's helpers, and at least one should be a broadaxeman if any round timber other than piling is to be used. If logged timber is to be used generally, all of the laborers must be axemen, and at least three should be broadaxemen.

For driving piles, a pile-driving crew may be temporarily organized from the above or may be organized for work with the

pile driver only. The latter method is recommended when a driver and crew can be kept continuously busy on various jobs.

Pile Driver Crew—
1 foreman,
1 engine runner,
1 fireman,
1 winchman,
1 leadsman,
3 groundmen,
2 pile hookers and trimmers,
1 teamster.

If all or part of the timbers are to be logged, a logging crew may be organized from the bridge crew and the logging completed before the work of erection begins. Generally, however, it is better to organize a small logging crew of men specially qualified to log and deliver at the site the sticks required by the foreman. The size of the logging crew will depend upon the amount and the nature of the logging required.

Logging Crew—
1 sub foreman,
6 axemen,
2 teamsters,
1 cook (if camped away from the bridge crew).

Additional teamsters will be required if there is a considerable haul to the bridge site. In certain localities logging may be economically accomplished, using the pile-driver engine instead of teams for dragging out the logs.

An engineering party consisting of one instrument man and two rodmen should be available at the bridge site when needed. Frequently, in highway work, the foreman will be able to run a level, and if given the proper instruments he can give his own cut-offs, etc., using members of his crew as rodmen. Alignment can usually be secured by eye if the locations of the approaches or abutments are given.

The equipment of the crews depends so largely upon the circumstances of particular cases that a standard equipment list is of value chiefly as an aid to the memory and to suggest other and special needs. A well equipped crew will usually have the articles named in the list below:

Bridge Crew Equipment—
1 pile driver complete, with additional rope, cable and wrenches.

6 snatchblocks and heavy manila rope (about 1¼ inches diameter),
6 pile hoops: two 12 inches, two 14 inches, and two 16 inches,
1 team and wagon,
2 log chains,
1 contractor's dolly,
2 four-line blocks and falls with heavy manila rope,
1 100-foot steel tape graduated in feet and inches,
3 carpenter's tool chest sets,
3 additional carpenter's squares,
2 carpenter's levels,
2 additional braces with assorted bits by sixteenth sizes from ½ to 1¼ inches,
2 ship augurs each size, 1 inch to 3 inches by quarter sizes,
6 cross-cut saws (1 one-man, 5 two-man),
2 rip saws,
1 saw setting outfit,
1 set of files, assorted grades of fineness,
1 adjustable boring machine,
2 sets framing chisels, assorted sizes,
2 carpenter's slicks, about 3 inches,
2 carpenter's gouges,
4 carpenter's hammers,
6 2-foot carpenter's rules,
Lumber, crayon, pencils, cord, etc.,
1 grindstone,
2 oilstones,
12 whetstones (for axes),
1 small forge,
1 anvil,
1 set blacksmith's tools (hammers, chisels, nippers, tongs, horseshoeing equipment, etc., etc.),
1 ratchet drill with assorted twist drills,
1 set taps and dies,
6 double-bitted swamping axes,
1 poleaxe,
2 broadaxes,
2 adzes,
2 striking hammers, about 6 and 8 pounds,
1 sledge hammer,
1 crow bar,
2 pinch bars,
1 wrench for each size of nut for bolts larger than ¾ inch required by plans,
4 monkey wrenches, two 8 inches and two 18 inches,
2 Stillson wrenches, 18 inches and 36 inches,
4 picks,
6 long-handled shovels,
6 peavies,
2 jack screws, about 2½-inch thread,

2 plumb bobs,
1 camp equipment, complete,
1 Wye or Dumpy level } If foreman is to do his own instru-
1 level rod } ment work.

The logging crew will require a wagon or sled and two log chains to each team, two double-bitted axes for each man, one peavy for each man and may require camp equipment.

The above list will be found reasonably complete for general cases, but special cases will require additional quantities and tools not mentioned above, while some articles listed may not be required.

For the pile drivers required in highway bridge construction, a double-drum, two-spool hoisting engine, using a 2,000 to 3,000-pound hammer, will usually be most satisfactory. Leads about 30 feet long should be provided and the frame should be long enough to give an overreach of at least 18 feet.

Erection. On the arrival of the crew at the site and the completion of the camp, the logging crew should be organized and started upon its work. A framing yard as near the bridge as practicable and convenient thereto and to the delivery point of the materials, should be cleared. The bridge site should also be prepared. All framing other than the simplest of right-angle notching should be done by the carpenters, and usually in this yard. Helpers should be assigned to the framing yard to move timbers, assist in sawing, boring and similar rough work. Carpenters should not be required to move timber, etc., except when absolutely necessary. The pile-driver crew should be organized, the driver set up and moved to position. Laborers not otherwise employed should be engaged in clearing the bridge site or drawn from other work for the purpose if necessary.

Framed trestles may usually be framed at the point of erection. If the members are too large to be raised by hand, a small gin pole at one end of the sill can be erected to raise them. If many bents are to be erected, a small force should be assigned to complete all of this work. Framing and erecting bents and placing sway and longitudinal bracing can be performed by laborers, one of whom should be a carpenter's helper.

Piles may be successfully driven in almost all soils except solid rock, quicksand, and large boulders. In quicksand they can frequently be jetted or jetted and driven into place. In boulder formations, unless the boulders are large and close together, the piles can be driven, but not to accurate position. In driving in boulders or gravel the piles should be shod with a conical shoe of $\frac{3}{8}$-inch sheet steel. Many types of shoes are used with varying degrees of

satisfaction, but the writer has found the shoe shown in Fig. 9, Plate I, the most satisfactory of several types used.

A pile, shod and hooped ready for driving, is shown in the foreground of Fig. 10. In practically all driving the piles should be hooped. The butt of the pile should be "dubbed" or beveled to receive the hoop, the hoop used being the next size smaller than the diameter of the butt of the pile. A good pile hoop can be made from $\frac{5}{8}$ inch or $\frac{3}{4}$ inch Norway iron, 3 inches wide, bent to a circular form and welded.

The conditions of driving determine the methods to be employed and are too varied to be considered here. The work may require simply the operation of the driver or may include the operation of

Fig. 10. Gulkana Bridge, Gulkana, Alaska, during Construction. Piles for South Abutment Driven. (Note piles shod and hooped, in foreground.)

a considerable floating plant. In most cases, however, the driving will be from the ground, from bents already driven, or from false bents driven for the purpose of supporting the driver.

If frozen ground is encountered it must be thawed by driving thawing points of the proper length into the ground before attempting to drive the piles. A thawing point consists of a hollow iron bar, usually 6 feet to 9 feet long with a wedge-shaped point. The interior cavity opens through the point and has a steam hose connection near the base. The point is driven into the ground as far as possible by hand, and steam at boiler pressure is forced through it. As the ground below the point thaws, the point is driven deeper, until the penetration desired for the pile is obtained. The point is then withdrawn and the pile driven. Care should be taken to see

that the point remains in the ground long enough to thaw a radial distance of 2 feet or more around it, or the pressure of the frozen walls will not permit the displacement of the interior thawed material by the pile. This condition may be indicated in ordinary driving by the springing of the pile as in driving in quicksand.

There are few points of difficulty in the erection of crib or pile and crib piers that belong to general cases. In the construction of piers with piling, the latter should all be driven before any portion of the bracing is placed, as the spacing actually secured for the piles may slightly modify the exact location of the bracing or cribs. Fig. 11 shows a pier of this type with the nose completed and the piling driven for the rectangular crib. If the conditions of driving are such that the piles can not be accurately placed the wall piles of a pile and crib pier should be driven well inside the wall line and the wall may be made to bear against them by blocks when necessary.

Trestles do not require false work in their erection. Frequently king-post trusses can be erected without false work below the chord line by using the chords as beams to support the trusses during erection. False work from the ground may frequently be desirable, but if at a great height the trusses may be erected from a temporary single lock bridge (military type) between the abutments.

Larger bridges usually require false work built from the ground. This false work consists of a series of two-storied trestles with the caps of the upper and lower stories 2 or 3 inches below the positions to be occupied by the upper and lower chords, respectively, in the unloaded bridge. These bents are ordinarily two-post bents. They are usually placed at the panel points of the truss, or near them if such location would interfere with framing. They must be given sway bracing and longitudinal bracing. In the interests of economy, the false work may be of second-hand lumber. If from timber cut from the forest, it may well be dry timber such as can frequently be cut from burned areas. These remarks are not to be understood, however, as advocating the use of *defective* timber in false work.

The vertical posts of the false bents may be piles or false piles (i. e., piles set by hand, like posts) or simply posts with or without a mud sill. If the driver has to be moved along the bridge site on false bents, these bents should be so driven that they can be used in the false work. The chords are raised to positions on this false

work and are blocked to accurate position and completed throughout. The blocking permits adjustments necessary from time to time because of the settlement of the false work and facilitates the removal of the false work after the bridge is completed. In light bridges the chord members can usually be raised by hand. In heavy structures, a gin pole may be erected with and supported by the false work near the center of the span for use in raising heavy pieces. In Howe trusses, the diagonals are next placed and then the verticals, the nuts on the latter being tightened to their positions. The wind bracing is next placed and tightened. The false work should be removed, commencing at the center of the span and working both ways. The floor is then completed, and after its

Fig. 11. Gulkana Bridge, Gulkana, Alaska, during Construction. Piles Driven for River Pier and Nose Completed.

completion the nuts of the verticals and wind bracing should be retightened.

The order of the placing of the members of combination bridges depends upon the arrangement of the members upon the pins, but the method of procedure is similar. In the erection of the cable type of combination truss, as previously mentioned, the verticals and top struts take the place of the upper story of the false work and the roadway bearers are the caps of the lower stories. In construction, the roadway bearers should be placed about 1 inch below their positions in the unloaded bridge and are not spiked to the false work bents. In place of spiked caps these bents may be cross braced at their tops by waling pieces. The diagonals are tightened

with blocks and tackle and fastened with cable clips. When the diagonals are properly tightened, the bridge clears the false work. The cables in the wind bracing are continuous through consecutive panels and are tightened by twisting a rack stick through loops at the panel points as shown in Fig. 12, Plate I.

III. COSTS.

The costs of a bridge, whether estimated or recorded, are classified, practically, under the various headings of overhead, engineering and superintendence, foremanship, plant and equipment, materials of construction, labor, etc., and the several headings are made to apply to various classes of construction, such as preparing site, logging, erecting piers, false work, framing, superstructure, erecting superstructure, etc., etc. In estimates for funds, an additional item for contingencies varying with the probable exactness with which the construction conditions can be foretold is generally included.

In general, all costs are of two main classes; i. e., costs of material and costs of labor. When ordinary commercial products are used, the cost of material at the bridge can be very accurately determined from market prices and hauling charges. Overhead, engineering, plant charges, etc., depend upon the organization engaged in the work. Plant charges depend upon the cost of renting or upon the purchase price and value of the plant after use. This value may be a sale value or, as is generally the case, it may be an estimated value obtained by deducting from the cost price an assumed depreciation when the plant is to be used in other work. Superintendence, foremanship, etc., depend upon the organization, rates of pay and the amount of work involved, and will usually fall between the limits of 6 to 12 per cent of the labor cost.

All of the above items may be computed from the conditions belonging to each case, the accuracy of the computations depending upon the completeness with which these conditions are understood. In the comparison of costs of alternative designs, the cost of materials will be proportionate to the quantities involved, the cost of the plant can be computed with considerable accuracy, or is fixed by arbitrary rule, and the overhead superintendence, etc., are fixed percentages of the labor cost. The labor cost is, then, the chief item of uncertainty, and the comparative accuracy of the estimate depends mainly upon the accurate determination of this item. It varies with the excellence and the wages of the laborers (including mechanics and artisans) available.

The writer will not attempt to cover the general field of estima-

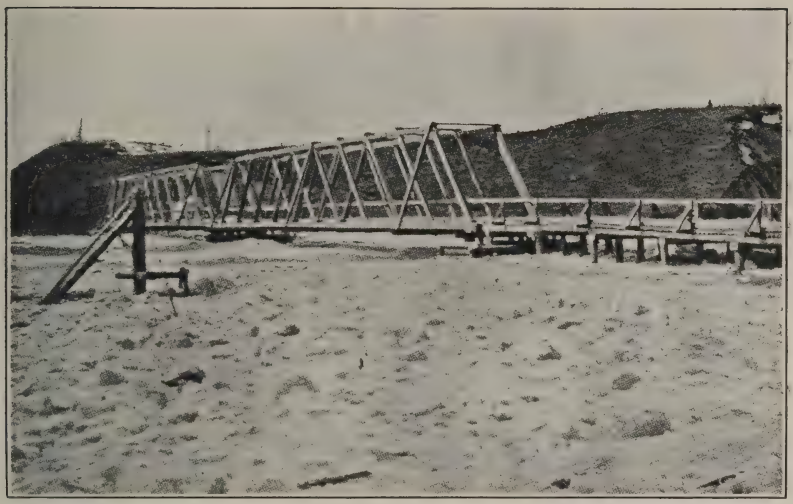


Fig. 14. Two 150-foot Wooden Truss Spans of Douglas Fir. (From Annual Report of Board of Road Commissioners for Alaska, 1913.)

Fig. 15. Wooden King-post Truss, Valdez-Fairbanks Road. Thirty-foot Span of Alaska Spruce.

ting and cost keeping, but will confine these remarks to a general analysis of the cost records taken from his own experience, with a view to giving some foundation for arriving at the probable costs of similar construction in other places. For ready application to localities with varying wage scales, the unit costs are given in hours of labor and not in dollars and cents. The Alaskan laborers are excellent, being generally skillful, resourceful and willing. They are, however, frequently handicapped by the unfavorable climatic conditions under which they work, and their average performance is believed to be about equal to that of the average white American laborer in the United States.

The costs given below are in hours of the particular class of labor adapted to the kind of work under discussion; e. g., framing by carpenters, logging by axemen, blacksmithing by a blacksmith, etc. Maximum, minimum and average costs are given for the several classes of work. The maximum and minimum are, respectively, the average unit costs of work of the class indicated on the bridge for which such average unit cost was greatest or least. The average is a general average of the unit costs of all the bridges on which accurate records of the particular class of work were kept.

Cutting Timbers. Native spruce, sticks suitable for piles, crib logs, truss members, etc., each stick about 20 feet long, diameters from 8 inches to 18 inches; cost includes trimming off branches, etc.: cost per linear foot:

	Hour.
Maximum	0.22
Minimum	0.04
Average	0.18

Hauling Timbers. (Sizes as above), all hauling on snow, hauls averaging about 1 mile, per linear foot:

	Hour.
Maximum	0.21
Minimum	0.09
Average	0.14

These costs include labor of loading, unloading and driving teams, but do not include teams. The number of teams depends upon the condition of the roads to such an extent that their record is of little value in estimating similar costs in other localities.

Framing and Erecting Cribs. Per linear foot of timber used:

	Hour.
Maximum	0.09
Minimum	0.08
Average	0.09

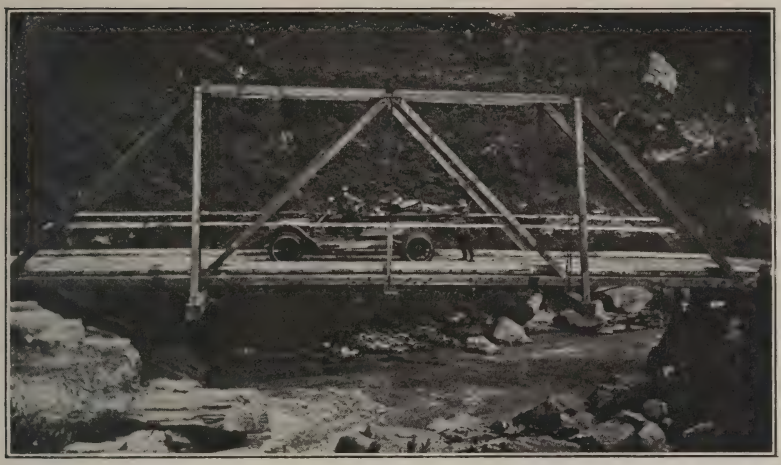


Fig. 16. Ptarmigan Drop Bridge, Valdez-Fairbanks Road, Alaska. Sixty-foot Span of Douglas Fir.

Fig. 17. Tonsina Slough Bridge, Chitina Fairbanks Road, Alaska. Sixty-foot Span of Alaska Spruce.

The above were all cribs of round logs with upstream noses only, but included both simple cribs and pile crib piers. The cost of erecting cribs of sawed lumber is doubtless somewhat less *per linear foot* of the timber used.

Filling Cribs. Per cubic yard of stone (crib measurement):

	Hours.
Maximum	5.90
Minimum	3.20
Average	4.58

In no case did the radius of the haul exceed 1 mile. The stones were picked up from river bars and piled before snow fell and hauled on snow in sleds. The costs are costs of labor in piling, loading, unloading and driving. The hours of teams were about half the above.

Driving Piles. Per linear foot of penetration:

	Hour.
Maximum (gravel with large boulders)	0.77
Minimum	0.50
Average	0.63

The above records include driving in many kinds of material and under varying conditions. They include all of the operations of the driver crew: moving, driving false bents, trimming and shoeing piles, etc., etc.

Thawing for Piles. Per foot of penetration:

	Hour
Maximum	0.25
Minimum	0.25
Average	0.25

The above are labor costs of firing boiler and handling points, but do not include fuel. A 12-horsepower boiler will set 5 points 9 feet deep in about 5.5 hours. Experience recorded by persons engaged in placer mining in Alaska indicates that three-fourths of a cord of dry spruce wood will set 5 points 9 feet deep under average conditions in gravel.

Trestles. An estimate for trestles may be made from the items given. Below are the labor costs of three trestles for which the records are not sufficiently complete to give the costs of the various divisions. The costs include all labor of building the trestles of round timber; i. e., getting out all timber, preparing site, driving piles, erecting trestles, flooring, railings and adzing deck. The trestles were approximately uniform, height about 8 feet, width 12

feet, but the difficulties of securing good timber and of driving piles varied greatly. The cost given is per linear foot of completed trestle. It includes foremanship, blacksmithing and all field expenses of the crew chargeable as labor.

Per linear foot of trestle:

	Hours.
No. 1. Long haul for timber, difficult driving in gravel with large boulders	16.6
No. 2. About average conditions	8.0
No. 3. About average conditions	7.3

Laying Floors (stringers, deck, guard and hand rails). Round

Fig. 18. Sixty-foot Truss, Valdez-Fairbanks Road, Alaska. Alaska Spruce.

timbers throughout with floor adzed after laying. Cost of completing the same amount of flooring that would be covered by 1,000 feet, board measure, of sawed lumber:

	Hours.
Maximum	40.42
Minimum	21.45
Average	26.10

Adzing costs about .05 hour per square foot of flooring surface and adds about 11.2 hours to the labor of laying the equivalent of 1,000 feet, board measure.

Original records of the costs of laying sawed stringers, decking

and railing are not available, but the writer believes that it should not exceed 8 hours per 1,000 feet, board measure.

Erecting False Work. One record only:

	Hours.
Per 10 linear feet of span	3.5

Framing and Erecting Trusses. Includes erecting and removing false work for bridges built on false work supported by the ice:

Per 1,000 feet B. M. in trusses—

	Hours.
Maximum (150-foot trusses)	64.42
Minimum	33.00
Average	51.38

All of the trusses were Howe trusses greater than 60 feet in span except the case giving the minimum, which consisted of one 89-foot span and one 50-foot king-post span in which the records of the cost of framing and erecting each truss were not separated. The maximum case is the only one involving spans as long as 150 feet. In two cases framing and erecting costs were kept separate, as follows:

Cost per 1,000 feet B. M., in trusses—

1. Four 75-Foot Truss Spans.

	Hours.
Framing	18.76
Erecting	26.40

2. Two 150-Foot Truss Spans.

Framing	23.18
Erecting	41.24

Blacksmithing and Foremanship. The labor of blacksmiths varied on the different works from 2 per cent to 5 per cent of the hours of all other classes of labor.

The time of foremen averaged 3.7 per cent of that of all other labor, excluding blacksmith, and was nearly constant owing to the uniformity of the size of the crews employed for all classes of bridge works.

The writer has no original data on the cost of framing and erecting combination trusses, but it is believed that the cost should not vary greatly from that of framing and erecting the equivalent wooden trusses, but should be slightly less.

Figs. 14 to 18 show several small and light wooden bridges built in Alaska.

Strategy and Grand Tactics.*

Strategy and Grand Tactics lie at the very foundation of the Art of War. Their full treatment would require an elaborate theoretical investigation of the principles developed by experience, and extensive illustrations from the examples of history. In the space available, I can only hope to bring out, in a very general way, the leading ideas grouped under these two heads, and to suggest some lines of thought in connection with the development of military science which may possibly be of value in the future.

It is not easy to frame a logically accurate definition of the term strategy, nor can we briefly indicate the limits within which it applies and the line which separates it from that portion of military science included under the head of tactics. The meaning of these terms will, I trust, become clear as we proceed. For the present, very general definitions will suffice. Strategy is the art of making war upon the map; tactics is the art of making war upon the ground, in the presence of an enemy.

In order to develop the meaning of these definitions, let us consider the circumstances existing in every war. In the first place, we observe that there are at least two opposing bodies of armed men, one of which is to be employed to effect by force some purpose which it is the business of the other to frustrate. Considering the army which is on the offensive, we see that it must be moved from some starting point to some other point where its object is to be effected. During its movements it is to be fed and clothed, and furnished with medical and other supplies; and should battles occur, its ammunition is to be replenished. Finally, at certain points of its progress,

*This lecture was delivered at Yale University in 1882 and again in 1883 by Capt. Chas. W. Raymond, Corps of Engineers, U. S. Army. It is here reproduced because of the clear and concise statement it gives of the essential principles of the subject matter and its present value and interest, notwithstanding the period which has elapsed since it was prepared. It is pertinent to the fighting now going on in the Eastern battle zone of Europe, and would be likewise to any likely to occur in this country. Contributed and edited by Maj. R. R. Raymond, Corps of Engineers.

it is to be maneuvered in actual contact with the enemy. Thus it is plain that the problem of war consists of three parts:

1. Where, when and how to move an army.
2. Where, when and how to supply it.
3. Where, when and how to fight it.

Strategy deals only with the questions, Where and When? the question, How? forms the subject matter of other branches of military science.

Examining further the circumstances of an army on the offensive, we observe that three things require consideration:

1. Where, when and how to move an army.
2. Where, when and how to supply it.
3. Where, when and how to fight it.

These I will now examine in order.

The place from which an army starts is called the base of operations. It is a portion of country adjoining the theater of operations where supplies and reinforcements may be collected, from which the army moves to take the offensive, and upon which it falls back if obliged to retreat. As it is difficult, and perhaps dangerous, to collect all the supplies at one point, several points are usually selected so as to provide in the best manner for the distribution of the depots. These points should be secure against sudden attacks. They should, therefore, be strong positions, well fortified, and connected by good communications.

A wide unfordable river, with both sides fortified, is generally regarded as the best kind of base. In case the army is compelled to assume the defensive, it forms a strong line of defense. It is secure against sudden attack, and it furnishes a good communication with the different points. It is evident that the selection of the base will depend upon the character of the war and the plan of campaign. When an army invades an enemy's country, the base is generally that part of the frontier separating the army from the theater of war. Where an army is defending its own country, the base is generally the capital and the country surrounding it. When the offense has the control of all the navigable waters separating the two countries, a portion of the enemy's own country, in which there is a good harbor, is frequently used as a base of operations.

We will now consider the point or points to which the army is to go. These are called *objective points*. These may be divided into two classes: *natural* and *accidental* objectives. A natural, or as

it is sometimes termed, a geographical objective, may be an important position, naturally strong or made so by fortification, the possession of which gives control over a tract of country, or is of value in subsequent military operations. Or it may be a great commercial center, or the capital of a country, the possession of which forces the enemy to sue for peace. Accidental objectives, or as they are sometimes termed, objective points of maneuver, depend upon the military operations themselves. The position of the enemy determines their location. Thus, if his forces are scattered, or his front much extended, the center of his position would be a good objective, since its possession would divide his forces so that his detachments could be attacked separately. If his forces are well supported, the flank nearest his line of communication is often a good objective, since, if it is forced back, he is in danger of being cut off from his base of supplies.

The route by which an army moves from its base to its objective point is called the *line of operations*. The roads passed over as an army advances are generally those taken when it retires or is driven back. In the latter case, they are called lines of retreat. The term lines of communication is applied to all the practicable routes connecting the different parts of an army in the field. Therefore, as an army advances, lines of operations become lines of communication. The safest and most convenient of these communications will generally be the central one; that is, the one leading from the center of the army back to the base. This is sometimes called the line of supplies.

The amount of ammunition expended by modern firearms is so enormous that it is generally impossible for an army to carry more than enough for a single battle. Hence it may be impossible, after a general engagement, to continue the contest, if the enemy has obtained possession of the army's communications, and the most disastrous results may ensue. The roads are filled with trains carrying forward supplies and returning with sick and wounded. Detachments and stragglers are scattered along them. All these fall into the enemy's hands, should he obtain possession of the roads. In addition to all this, the moral effect produced by closing the natural line of retreat of an army, so that its defeat in battle means nothing less than utter ruin, is still more to be dreaded. Hence the vital importance of thoroughly protecting these lines is apparent; and this is a problem which becomes more and more difficult of solution as the lines of communication increase in length.

The military problem considered under the head of strategy will now be readily understood. An army is a large mass of men covering in its movements a wide extent of country. Its operations require that it shall be more or less scattered and divided, and its concentration requires time. It must be continually fed and supplied. An army on the offensive must, therefore, either cut and hold the enemy's line of communications, thereby forcing its surrender, or it must be brought to the objective at such a time and in such a way as to engage in mass a portion only of the enemy's forces. The selection of objective points at which the results obtained will be decisive in their character, and the selection of bases and lines of operations best suited to the above objects, constitute the strategical problem of war.

The so-called "principles of strategy" are merely generalizations from the facts developed in the history of war. Hence the student of military science meets a serious obstacle at the very threshold of his investigations. In order clearly to comprehend the meaning of fundamental principles, he requires a knowledge of the facts on which they are based; in order to appreciate the true bearings of the facts, he requires a clear comprehension of military principles. From this it arises that some have come to the conclusion that this purely theoretical study is of no practical value. Now it is true that the principles of strategy can not be employed, like the fundamental principles of the exact sciences, for the sure deduction of accurate results, but they are of the greatest value to the advanced military student in grouping and arranging the numberless facts of the history of war.

Passing over the important subject of Logistics (or the art of moving and supplying armies), I now invite your attention to the subject of Tactics. I have already remarked in a general way that tactics is the art of making war upon the ground, in the presence of an enemy. In other words, it is the art of posting troops upon the battlefield, of bringing them into action and of conducting them while fighting. Strategy decides where and when to act; logistics brings the forces to the objective at the proper time; tactics decides the manner of execution and the employment of the troops. It will therefore be remarked that strategy constitutes that part of the science of war which does not change with the improvements in the details of the military art; for the principles of strategy depend solely on three unchangeable conditions: viz, that armies must be moved, that they must be supplied, and that they must be fought.

Tactics, however, is constantly changing with the general advance of military science; and this branch of the art of war is, therefore, by far the most interesting and the most suggestive one to the general student, who has not made a sufficiently extensive study to feel the need of broad generalizations.

It is often said that the introduction of breech-loading rifles and rifled artillery has completely changed the system of fighting. In one sense, this is an error. It is true that improvements in arms have produced considerable modifications in tactical methods, but these changes are merely the growth and development of principles known for hundreds of years. There are, indeed, periods when great changes seem to take place, but these periods are coincident with great wars, when the minds of people generally are turned to military subjects. At such times, many startling discoveries seem to be made, which have long been known, discussed and practised, but have escaped notice until forced into prominence by the course of events.

The study of tactics, then, is the study of the growth and development of military science, and it can be most successfully pursued by examining the condition of tactics at various times, and the changes which have occurred in passing from one period to another. Such an examination is far beyond my limits. I will, however, endeavor to indicate briefly the character and tendencies of the modifications which have taken place.

Going back to the history of Greece and Rome, we find the same causes then at work which produce alterations in tactics now. We find the undisciplined armies of Persia, composed of nations and tribes of warriors, overcome by the steady discipline and firmness of the Greek phalanx. But this same phalanx was overcome by the Roman legion. The essential feature of the phalanx was that it was a solid mass, generally formed in the shape of a wedge, for purposes of attack. The legion, however, was comparatively an open-order formation, and was so organized and subdivided that its various parts could be maneuvered independently and with considerable freedom. The legion was evidently far better adapted to circumstances of locality than the phalanx, which could only move effectively on even ground. In the phalanx, the keeping together of the entire body, whether in moving onward to bear down the enemy by its pressure, or in waiting to resist his shock by its inertia, was everything. In the legion, individual activity and the ease with which the smaller subdivisions lent themselves to every requisite

movement, gave to the entire machine the volition and strength of life. In connection with these ancient tactical arrangements, two points are to be noted. First, the effects produced by the actual shock of masses and the resistance offered by the inertia of these close formations to the impact of an attack; and second, the clear indication of a fundamental tactical principle: viz, that troops must be so formed that they shall be mobile; that is, readily maneuvered and engaged in conformity with the irregular topographical features of the battle-field.

When soldiers were armed with pikes and axes and shields, the best formation in which they could be arranged for action was that of deep columns; for, if opposed by a thin, extended line, the columns would penetrate it by their momentum, and make gaps through which the cavalry would pour to attack the enemy in rear. Again, the column, by approximating to a solid square, possessed the formation necessary to resist the charges of the men-at-arms. If suddenly assailed in flank, it readily offered on that side an impenetrable face, and maneuvers to turn and outflank an enemy were accordingly little thought of. Even archers drawn up many ranks deep could still use their weapons with effect. The line of battle, then, was a line of dense columns—a formation easily assumed from the column of march. Requiring but little practice or instruction, it was best fitted to the motley bodies which the different knights led to the field. Further, it is to be remarked that the column is the formation in which men are best maneuvered and controlled, for it is evident that when lines are extended the men are removed further from the commander's voice and eye, and their concerted motion becomes much more difficult.

The introduction of firearms resulted in a great change in fighting formations, for as two ranks only could discharge their pieces, to form on any greater depth was to sacrifice something of the fire of the line. At first only a part of the troops were armed with muskets, and the diminution in depth was only partial—the front line of battle still standing six deep. But when every man was a musketeer, the line was proportionately extended. Three ranks, however, were still considered necessary; the third, which could not fire, loading for the others, and being thought to add to their stability, an idea which prevailed in some European armies down to a comparatively recent period.

To deploy into extended lines demands more practice and precision in drill than to move in deep columns; but the training of

armies did not at first keep pace with these requirements. In the time of Marlborough and Eugene, deployments were effected but slowly; positions were deliberately taken up and fortified, and the opposing army, with equal deliberation, drew up and attacked in parallel order. There was little of what we call maneuvering; that is, the quick, orderly change of highly trained and flexible masses from one kind of formation to another or their transfer from point to point of a battle-field for purposes which might become suddenly feasible in the changing course of the action. Generals displayed their skill in choosing a position adapted to the numbers and composition of their army, and arranging the troops for its defense and in detecting points in the opposing line where the hostile leader had made blunders in his dispositions, and directing thereon the decisive attacks.

With Frederick the Great there arose a new order of things. He found to hand an army formed by Frederick William, his father, which was probably the most perfect military machine that had ever existed. He perceived that the secret of success must lie in turning to full account that power of maneuvering which his father's system had imparted to the Prussian troops, and which had been perfected under his own direction. The improvements he introduced were in the same direction—mobility and development of fire. The troops were formed in three ranks. The greatest care was taken to preserve correct distances and alignments, so as to enable every man to fire. The movements were always in open column, which, at the proper time, wheeled into line and advanced on the enemy. The cavalry was formed two deep, and trusted entirely to the effect of its charge, not to its firearms. The Prussian order of battle was invariably in two lines, cavalry on the wings and infantry in the center. The artillery was usually placed in heavy batteries on the front.

The very short range of weapons at this period enabled troops to get personally engaged after passing over short spaces. When the troops were once engaged, Frederick's tactics consisted in rapid firing and repeated charges with the bayonet. Time will not permit a careful examination of the methods employed by this great master of the art of war. It must suffice to say that in general his army moved in two long, open, parallel columns along the front of the position occupied by the enemy, seeking to place itself obliquely on one of his wings. At the proper time the two columns halted and wheeled into line. It is evident that troops to move in this manner must be most carefully drilled, otherwise the distances would be

lost and great confusion ensue. Such formations and movements could never take place in the presence of an enemy who can maneuver, or who is armed with long-range weapons, for the wheel into line must then be made at a considerable distance from the enemy, and a long advance in line under fire would be necessary.

The system followed by Frederick had many disadvantages. Among these are the slight depth of his line of battle, the want of reserves, the impossibility of properly commanding the troops when spread out in such long lines and columns. Frederick's successes were principally due, not to his tactics, but to the admirable drill, steadiness and maneuvering powers of the Prussian army. This army formed for many years the model for all others, and the tactics of Frederick are the basis of what are called "linear tactics." Every civilized army is still strongly imbued with the spirit and the method which Frederick William so diligently imparted to his troops in the parades of Potsdam.

The next great changes in tactics took place, along with so many other changes, in the French Revolution. In the great war which revolutionary France maintained along her extensive frontier numerous small bodies, separated by many miles, often acted with a certain independence of each other, though directed by one commander. To make them capable of independent action, these bodies were now rendered complete in all arms. Divisions were organized of 8,000 or 10,000 infantry, 1,000 or 1,500 cavalry and 12 guns under one commander, and trained to maneuver in concert. A faculty of independent action was thus acquired, but it led to diffuseness of movement and want of unity in design.

But the Republican armies possessed a tactical method which was an ample compensation for this defect. Before the Revolution important modifications in maneuvering had been introduced, which soon proved successful against troops trained exclusively in the system of Frederick. For mobility and the mutual support of all arms, in which the Prussians had formerly been so superior, were now on the side of the French. The armies of France were recruited with great numbers of conscripts, full of intelligence and enthusiasm; but these had to go into the field without the instruction and practice in close formations necessary to meet the trained battalions of the enemy. Therefore, they were thrown out in the form of skirmishers, where their intelligence and courage were rendered of avail without the necessity of acquiring precision and coherence. It was soon seen that these lines, or clouds, of skirmishers might be made extremely useful to cover formations, to deceive the enemy

and to conceal tactical enterprises. Light troops had, indeed, existed before, but they were regarded as of small account on the field of battle. Now, however, they became capable of opposing disciplined battalions, and further training enabled them also to take their places in the line of battle.

Thus appeared for the first time in war that most important factor in modern military operations—the skirmisher. The experience obtained by French officers in our own Revolutionary War undoubtedly had much to do with the adoption of this new system, but its leading ideas were advanced by Mesnil Durand as early as 1774. "Columns," said Durand, "mass the greatest amount of force in the smallest space, and give free movement to cavalry and artillery; but these columns must be linked together by thick chains of skirmishers. Infantry should have two distinct formations. Line is the best for firing, column for maneuvers and attacks. The shallow formation has a tendency to make men halt and fire; the order in column has a tendency to make men advance. When troops are deployed in two lines, with cavalry on the flanks, they are weak everywhere and incapable of the least maneuver. When, on the contrary, each battalion is in column, with skirmishers in the intervals, the cavalry placed in rear can easily and unexpectedly charge to the front. This order is strong everywhere. It threatens the enemy with the fire of the skirmishers, the weight of its columns and the charge of its cavalry. An army so formed can march and maneuver with the greatest ease and rapidity."

These views were in vogue at the French military schools when Napoleon was a student. They were well adapted to the peculiar character of the revolutionary armies; but they were by no means reduced to a system. Much was left to the individual intelligence and enthusiasm of the soldier.

After the assumption of the imperial title, Napoleon prepared for the invasion of England. In the large standing camps formed at Boulogne, and elsewhere on the coast of the English Channel, many changes were made both in organization and tactics. Conscription with substitution was introduced in the place of universal service. The more settled state of the country, causing many openings in civil life, drew off much of the intelligence that had previously been forced into the army. Nevertheless, this army still possessed many of the elements which gave the early Republican armies such great moral power. It possessed that firmness and steadiness which it takes many months for troops to acquire. Its campaigns definitely

established the superiority of the French over the Prussian tactics. It was the army of Austerlitz.

But such tactics require for their complete development thoroughly trained and educated officers for company and battalion commanders, well-disciplined troops and enthusiasm, skill and activity on the part of the men. Long wars sap discipline. Trained men who fall from wounds or disease are replaced by recruits. Experienced officers, whose places can not be well supplied, are killed or wounded. These causes soon began to tell on the French army, and it is interesting to observe their natural results in the tactical changes which soon took place. These changes were chiefly the concentration of great masses of artillery to crush a portion of the enemy's troops in position; the advance of heavy columns composed of entire divisions formed in mass. Such formations were adopted chiefly to keep the troops under the eye and control of superior officers, and partly to produce that feeling of confidence which the thronging together of masses of men is supposed to impart. All the French formations became deeper, less dependence being placed on the individual intelligence and firmness of the soldiers. These changes well illustrate the connection that must exist between the individual capacity of soldiers, their military training and the tactical formations in which they fight.

When rifles began to take the place of smooth-bore muskets, many proposals were made for changing tactical formations, and the effect of these weapons was much discussed. It was evident that infantry fire had become much more important than formerly, and that the formation of troops must be altered to give a greater development of fire on the one hand, and greater protection to the men when advancing on the other. Nevertheless, in the War of the Crimea, and in our own War of the Rebellion, the tactics employed were essentially those already described. Indeed, our own tactics were founded, with little change, on French *Ordonnances of 1831-1845*. It is true that, in our war, the use of skirmishers and open formations was probably carried to a greater extent than ever before, but this was due more to the character of our troops and the nature of the country in which we operated than to any other cause. Whatever slight changes occurred are insignificant in comparison with those following the general introduction of the breech-loading rifle, changes which have been brought into prominence by the experience of three great wars.

Looking back over the development I have briefly sketched, it is

easy to see what have been the marked features of its course. We see how, as military science advanced, the principle came to be more and more clearly recognized that troops must be so organized and trained as to be mobile—that is, capable of prompt and orderly maneuvering; and how, to this end, discipline became more and more important. We see how, as weapons have improved, there has been a gradual tendency to substitute for the shock of masses the shock of fire. We see how tactical formations have gradually developed and extended, first to give more effective play to the improved weapons, and finally because, under the fire of modern arms, the close formations can no longer exist. All these points will be more strongly illustrated when we consider the tactical conditions to be satisfied at the present day.

The modern era of tactics may be said to date from the year 1866, the year of the Austro-Prussian War. It is the era of the breech-loader. The introduction of this weapon into modern armies revolutionized tactical arrangements, but I need scarcely say that the principles of tactics remain unchanged. In their application, however, we are forced to make many modifications. In order to indicate these, I will endeavor to describe the course of a modern battle, commenting upon points of importance as we proceed.

There are really only two ways in which a battle can arise:

1. When one army is on the defensive, the other moving forward to attack it.
2. When the two armies meet, both being in movement.

In every case, the rôle of attacker or defender falls to one side, sometimes to each alternately; and it may vary in different portions of the field, the same army being on the defensive in one place and on the offensive in another. In whatever way a battle may come about, the order of march must be intimately connected with the action of the troops in line of battle. It must, therefore, be detailed so that the arms first required shall always be near the head of the column. If we consider the way in which battles begin, it will be found that the cavalry of the advanced guard usually find the enemy and report his position. In all probability, they will be driven in on the main body of the advanced guard, the commander of which must then take up the best defensive position possible to cover the deployment of the troops in his rear. If the enemy is holding a defensive position, the commander of the advanced guard will not be much troubled, because few generals will like to leave a carefully prepared position, give up too soon the advantage of acting on the defensive,

or move until the plans of the attack have declared themselves. In such a case, the commander of the advanced guard, knowing the enemy is to be attacked, must take up a position to protect and cover the advance of the main body. If the enemy is not holding a defensive position, he will probably be strongly pressed and have some difficulty in holding a point on which the troops in rear can form up to the right or left. Consequently, those arms of the service which can facilitate the formation should be near the head of the column.

It is manifestly the object of the commander of the advanced guard to find a position which will compel the enemy to develop a considerable force to attack him, and at the same time enable him to make a successful resistance. Such positions can generally be found. It is often desirable to occupy a small village or town, loophole buildings or form trenches, which will enable the advanced guard to remain in possession of the ground. Sometimes it is necessary to open passages to the right and left to facilitate the formation of the troops. A strong body of engineers should, therefore, move with the advanced guard. It is always well to strengthen some points to serve as rallying places in case of a reverse.

But no position can be properly defended without artillery; hence, a strong force of artillery should accompany the advanced guard. This should be used boldly to search out the enemy's position and compel him to unmask his guns. The artillery commander of the advanced guard occupies a very responsible position. He must not only find positions for his own guns; he should, from the fire of the enemy, be able to select places for the corps' batteries as they come up, and to facilitate this object he should place his batteries so as to leave room for others to take position in support of them.

To understand how the main army comes into position and how the actual fighting progresses during the battle, we must consider what have been called the "fighting tactics" of the present day. We have seen that, from the first, there has been a gradual tendency to extend the front of an army occupying a given position, or to reduce the number of men occupying a given piece of ground. Now concentration is undoubtedly strength, but we must clearly understand what is meant by concentration in its military sense. It means the assembling at a given point of a sufficient number of military units to deny the use of that point to the enemy, if we are acting on the defensive; or to drive him out, if we are acting on the offensive. But if that sufficient number be exceeded, concentration not only deprives other points of their proper defense, but it becomes

positively injurious, because it prevents the proper use of the men's arms. This gives us a limit to concentration dependent upon the action of one side; but it is further limited by the effects of the fire of the opposite side upon densely concentrated masses. Hence it follows that a certain number of military units are best adapted to the attack or defense of any given position, and it is undesirable to exceed or reduce that number.

The tendency to which I refer has been very gradual. Gibbon long ago remarked "that an army whose discipline and determination will allow of its fighting in the most extended order has always been victorious;" but during the last twenty years the great improvements made in firearms have produced a sudden expansion of the front occupied by armies. The number of troops considered necessary for the occupation of a position of given length is now much less than formerly, because the power of the individual firearm is now so much greater.

There has also been developed during recent years a tendency to occupy long lines by holding certain points strongly, and leaving others, comparatively speaking, weak. This tendency has been much strengthened by the power of resistance that the modern rifle gives troops when acting on the defensive, and the fact that even a small body of men can not be turned out of a position until regularly attacked, thoroughly shaken, and demoralized by fire. It follows from these considerations that the line of battle spreading over extended front and broken up into numerous distinct and separate bodies, when once actually engaged, can rarely, if ever, move to the right or left. Maneuvering under fire, always difficult, may now be deemed almost impossible. Once troops are really engaged, it would appear that their movements must be either forward or backward. All that can be done with the line is to bring up fresh troops from the rear to feed the fight in front. Under the fire of modern artillery, backed by small arms, maneuvers are impossible.

It is, therefore, apparent that the success of operations in war must primarily rest on the action of small bodies. Good previous strategical movements, a correct formation of the order of battle and a proper selection of the points to be attacked, will increase the importance of the success when obtained; but the *actual* success must ultimately depend on the correct handling of small bodies of men. The question how a small body of troops shall be handled in actual fighting has now become of far greater importance than formerly. This is one of the marked features of modern fighting, and one which

must be carefully considered. Under existing circumstances, a battle is a series of small combats in which bodies of troops, perhaps not greater than a brigade, are engaged. Each of these bodies must be tactically complete, must work for one object, and seek to carry one point.

You will now understand why I have passed over in silence all those great maneuver-tactics of armies which were so elaborately investigated by military theorists of earlier days. They are no longer of importance because they are no longer possible. The circumstances of conflict of the brigade form the vital problem of modern war.

Now what are the consequences of this division of battles into numerous small combats? One consequence is that the losses of the different sub-divisions of armies are very disproportionate. Military statistics show that the relative proportion of killed and wounded to the total force engaged has by no means increased since the introduction of improved weapons. But if modern battles be analyzed, it will be found that the great mass of the loss takes place at one point and in one corps, to whose lot it has fallen to capture or hold certain special and decisive points in the field of battle. It is a consequence of improved weapons that, if a lodgment is made on the key of the defender's position, the results extend to the flank more rapidly and to a greater extent than formerly. Hence every effort should be made to discover such points and to strengthen and maintain them so long as they are held. Attacks on the places of minor importance produce but little effect.

Another consequence is that numerous flank attacks can be made. At first sight it appears that an army, having only two flanks, can be subject to only two such attacks. But the fact is that each distinct portion of the field will offer such points for attack, which become more marked by the swaying backwards and forwards of the general line.

It further results that the formation of a reserve for the whole army is no longer admissible. The extension of front is so great that the reserve, even if placed in the center, could hardly reach the flanks in time to be of use. We, therefore, arrive at a general rule that no body of troops should ever engage without forming its own reserve. These reserves for the fractions of an army must not be massed together, but must be numerous, of a size proportionate to the object to be attained, and so disposed that they can act without delay when needed.

In addition to a reserve, there should be a second line. In small affairs a single supporting body might be sufficient, but no commander should ever be without a force held in hand. The moment his second line and his reserves have passed out of his control, his power of directing and ruling the fight is gone.

It has been said that in the future no direct attack can possibly be made on troops in position, and that all attacks must be made on the flanks. Experience, however, clearly shows that flank attacks can always be met and defeated unless supported by front attacks. It has been pointed out that troops in attacking invariably spread out to the flanks, thinning the line in the center. This appears to be caused by an instinctive feeling that the best way to avoid an enemy's fire is to equal or exceed his front. Ultimately, however, a flank attack always becomes locally a front attack, for the defense always receives it either by throwing back a portion of the line or by bringing up a reserve.

Modern artillery compels troops attacking to quit their column formation (in which marches must be made) much sooner than formerly. Modern small arms also produce effects at longer ranges than formerly. Hence the problem of the attack is as follows: There is a certain space of from 1,500 to 2,500 yards swept by fire, the intensity of which increases as the defensive position is approached, and this space must be crossed.

Thus it follows, not only that greater use must be made of skirmishers, but also that the main portion of the fighting must be done by men in extended, not in close, order.

But no matter how devoted a skirmish line may be, and no matter how gallantly it is led, it will in a short time expend its force. It appears that in action an advancing body has stored up in it a certain amount of energy. When this is expended, it can do little more. It has a tendency to halt and consider it has done enough, and to look less to the front than to the supports behind it. This is the moment when the supports must be forthcoming. They can see their own men in front, and as far as they have gone the supports can go. The mere natural emulation of men will carry both on farther. Thus the fight in front must be incessantly fed from the rear, and hence the troops first advanced should be a portion of the body in rear. Gradually working on in this way, men may reach a point near enough to the enemy to make their fire tell heavily upon him; and what is still more important, be so close that any attempt at a change of position on his part (which means a diminution of

fire) will enable a forward movement to be made, or the supports to be brought up in a closer formation. The front, therefore, of all attacks must not only be covered by skirmishers to feel the way and drive back the enemy's skirmishers—the fight must be fed by continuous waves of skirmishers pushing on in advance of one another, and so passing over the ground swept by fire.

But the men sent forward must not be sent in driblets. If a handful of men be pushed on, and then another handful when the first are beginning to be demoralized, it will simply subject the troops attacking to being beaten in detail. Such a system is one that courts disaster. It must always be borne in mind that whatever attack is determined on should be made with a force sufficient to give a reasonable hope of success. A successful attack always saves life, for the men making it cross the shot-swept zone only once. If beaten they must cross it twice, once in advance and once in retreat. Men should understand that, when well under fire, it is much safer to go forward than backward.

I have remarked that the troops first pushed forward must be a portion of the same tactical body as those immediately in rear. This brings up the question, What is meant by "lines?" From a simple line of two ranks, it is impossible to feed a fight or support the skirmishers; hence the term "first line" must not be supposed to mean either a line of skirmishers or a two-deep line, but the troops actually fighting in the front. If a brigade of three battalions be taken as an example, one battalion being skirmishers, one in support, and one again behind at a further distance, the whole forms the first line, the general officer's command extending from front to rear over a far greater extent of ground than formerly.

The second line of troops may, until required, be formed in any way that is most convenient and best suited to the ground. The brigade or division commander can no longer accompany a line in its advance which, from its length, it is impossible for him to supervise. He must watch the action of the troops in front, and support them by ordering up fresh men from the rear. Ground to the front is often won by some company officer who has taken advantage of surface irregularities and pushed up when he is sheltered from fire, or when the fire of the attack has told heavily, reducing the number of defenders and shaking their *morale*. Watching for these movements, the brigade commander can push up fresh troops to hold the ground thus won and to support those who have won it.

To recapitulate, it appears generally that troops should be formed

for attack in at least three bodies; that these should work under one direction for one object; that they should be closely linked together, and that each should be a considerable force. A company is far too small to be so divided, hence the battalion must be taken as the attacking unit, if any real work is to be done.

The function of the skirmish line is to crush and overwhelm the enemy with bullets. Indeed, the whole essence of modern fighting lies in this maxim: Seek to overwhelm the enemy with fire and shake his *morale* until his fire becomes so feeble that he may be attacked. The object of the supports is to fill up gaps among the skirmishers and so sustain the intensity of their fire. The duty of the main body is to advance the moment the enemy is sufficiently shaken, and drive him out of his position.

The distances between these bodies must be governed partly by the work they have to do, partly by the trajectory of the arms in use, and partly by the character of the ground. In order properly to feed the skirmishers, the supports should be from 200 to 250 yards in rear. The main body should be able to reach the skirmish line in about five minutes—that is, they must be from 500 to 600 yards off.

But we must now return to the original question: How shall the shot-swept zone be crossed?

Leaving out of consideration irregularities of ground, this zone is swept in different parts by fire of different kinds:

1. The artillery fire, which may be said generally to begin at about 2,500 yards.
2. The unaimed or random fire of infantry, which begins to tell at about 1,100 yards.
3. The aimed fire of infantry, which begins to tell at about 600 or 700 yards.*

Artillery defending a position does not fire so much on the advancing infantry in the early stage as on the attacking artillery. Moreover, the loss it produces in action is comparatively very small, being only about one-twelfth of the loss caused by musketry. Hence this fire may generally be disregarded at the beginning of the attack.†

The unaimed infantry fire produces many losses, but its effects may be much lessened by observing where it falls most, and avoiding those places.

*It should be recalled that this lecture was delivered over 30 years ago, hence the distances given are not what would be stated to-day.

†Modern field artillery and machine guns are much more deadly than the artillery herein referred to.

The fire that causes heavy losses, and checks advancing troops, is undoubtedly the aimed fire of infantry. The quicker the space separating the place of first deployment and the enemy's position is crossed, the better. But if crossed on the run the main object of the skirmish line, the crushing of the enemy with bullets, would not be accomplished. Hence this is not desirable, even if it were possible to run over the 2,000 yards without stopping. Experience shows that until the attacking troops reach the zone of dangerous infantry fire, or about 1,100 yards from the position, they should if possible advance without firing, and as quickly as they can without disorder. When the dangerous zone of infantry fire is reached they will begin to suffer; but their power of inflicting loss upon the enemy then begins, and at this stage the skirmishers should open fire. Bearing in mind that the function of the skirmish line is to overwhelm the enemy with shot, and that the men must expose themselves as little as possible, it appears that from this point the advance may best be made in a succession of short rushes, lying down, firing rapidly, and again running on. These rushes should not exceed about 60 yards, in order that the men may not become exhausted or get out of breath. The rush is made by one company at a time, the other deployed company of the battalion covering the advance with its fire. The company commander selects the place to which he is going, points it out to his subordinates, and at a given signal rushes to the front, his men following. When he reaches his new position, he gets his men under cover as quickly as possible, opens fire at once, and fires steadily and deliberately until the other company rushes. Then he should fire a few rounds as quickly as possible to cover its advance.

But no fire, however heavy, will drive good troops out of a position. As long as they have ammunition they will remain until compelled to retire by the actual advance of superior numbers. Hence all fire must be regarded as simply paving the way for getting possession of the enemy's position. In this respect, modern weapons have made no difference whatever. A formed body must be placed in the position, and maintain themselves there, in order that victory may be achieved.

Thus far our attention has been confined to the attack. We must now consider how the defense is to be conducted. The defense is generally supposed to have gained great advantages over the attack by the introduction of modern weapons. This view we have seen to be in some degree erroneous. The following advantages have been claimed for the defense:

1. Their fire is delivered by men in position, who can take a more

correct aim. But it must not be forgotten that the assailants move rapidly and the defenders are always firing at a running target.

2. The defense is better covered than the attack, suffers less loss, and is consequently cooler and less excited. But the concentration of fire on one spot so demoralizes a defender that he can not be calm. His dead and wounded can with difficulty be removed, while as troops advance, they leave their dead and wounded behind.

3. The ground may be better studied by the defense than by the attack, so that a better advantage can be taken of its conformation, and additional force may be obtained by removing men from where they are useless to important places. This is a real and decided advantage, and can only be met by a superiority of force on the part of the attack.

For the correct holding of a defensive position, a heavy artillery and infantry fire should be brought on the attackers at the very beginning of the action, and a few powerful reserves should be held in hand to drive out any troops obtaining a lodgment within the general line.

But no defense has a chance of success that does not contemplate offensive action when opportunity offers. As I have already said, there is a tendency on the part of the assailants to avoid the center of the defender's position, from which the fire is usually heaviest, and to turn off to the flanks, seeking out places less thoroughly swept by fire. The gaps left by such movements are the points favorable for the defenders to attack, and thus threaten the enemy's advance on the flanks. But in making such forward movements, the greatest care is requisite that the main position be not lost, and hence arises another necessity for reserves. Generally the supports and main body may be much nearer the skirmish line than when attacking. They should be placed so that they may escape fire as much as possible; but, especially in the defense, it must be remembered that a small reserve that arrives when it is wanted is better than a strong one that arrives too late. Early deployment and a long advance in extended order exhaust an assailant. The defender must, therefore, seek to make the attacker deploy as soon as possible. This is perhaps better done by bodies of cavalry and artillery pushed well out to the front and supported by infantry, rather than by long shots from the main position, which are not very efficacious, and reveal the position held by the defense.

I must now refer very briefly to the use of the other arms. The great range and accuracy of modern guns enables a large number to be concentrated on a given spot. This is very favorable to the de-

fense. The attack also gains by it, for the guns can support the advancing infantry almost to the moment of actual assault. There seems to be no reason why all guns should not be brought into action as soon as possible, and remain until the close of the engagement. The side attacking usually establishes a cross-fire on some portion of the enemy's position. If this be a village, the fire will soon shake the buildings so as to render them indefensible. At the moment when the enemy appears to be leaving the village the infantry should advance. To prevent the enemy from returning the artillery should continue to fire until its fire is masked. The guns should then turn their attention to the enemy's artillery, which has probably been playing chiefly on them, although the advancing infantry have doubtless come in for their share. There are two general rules for the use of artillery: it must act in large bodies, and it must seek to bring a converging fire on the point aimed at.

As for the cavalry the history of recent wars shows that its duties are, perhaps, more important than ever, not only in covering the advance of an army, but also on the actual field of battle.

One of the most important duties of the cavalry is to keep the army acquainted with every movement of the enemy;* to harrass him and conceal the movements on their own side by keeping a strong curtain of posts constantly in advance; to connect the various army corps moving together, and keep each informed of the action of the others.

It may be questioned whether the charges of large masses of cavalry, such as were executed during the wars of the early part of this century, can now be carried out, except for some very important object which warrants the great loss that must ensue. The action of cavalry on the battle-field appears now to be limited to watching the flanks, checking the action of hostile cavalry or attacking broken and repulsed infantry.

And now, in concluding these hasty and inadequate remarks, it will be well for me to invite your attention to a few points of importance to us. We have seen that, from the first, progress in tactics has been of a two-fold character. The shock of masses has gradually but surely been replaced by the shock of fire, until to-day it has no place on the battle-field except to overwhelm a demoralized and broken enemy. Similarly, we have seen that the mobility of troops has gradually increased; but here I must call your attention to a very important point. While recent improvements in weapons have produced a great increase in the mobility of small

*This duty now falls largely to the air scouts.

bodies, they have had exactly the opposite effect upon large bodies of troops. Such bodies pass out of individual control and cease to be capable of maneuvering at a much earlier stage than formerly, for the terrible effects of modern musketry compel them to extend over long lines while they are still far from the enemy's position. Tactically considered, this is really a step backward, and it constitutes one of the most perplexing difficulties of modern war.

The expression "loose order" is often applied to the modern method of fighting in extended lines. The term is an unfortunate one, for it seems to imply a careless, haphazard method of action. In point of fact, such a method of fighting demands not only greater individual exertion and intelligence, but far higher and more complete instruction on the part both of officers and men. We have already noticed how, as the moral value of the French army diminished, their formations became denser and more compact from necessity of keeping the men more closely under the supervision of the superior officers. The converse of this is true, that to enable men to fight in individual order, not only must the officers be better trained, but also the privates themselves. The real result of the improvements in weapons is not to introduce a loose, careless method of action. It has rendered fighting far more difficult—rendered it necessary that much of the care formerly bestowed upon the *ensemble* of a dressed line or perfect column should be now devoted to the individual man.

Our own people are probably better adapted to the modern method of fighting than any other people in the world. They are brave, intelligent, self-reliant, full of resources and quick to adapt themselves to circumstances. These are the exact qualities needed for the individual fighting of modern war. Should we be called again to take the field, our great want will be a body of thoroughly trained and instructed officers. For we have seen that hereafter the actual conduct of battle must rest, not as heretofore upon generals and colonels, but upon the commanders of battalions and companies. Nor can we afford to learn by experience. Errors which, twenty years ago, involved slight consequences and might readily be retrieved, would now be followed by the most disastrous results. To our educated young men we must look for officers, in case of war; and I feel that my brief remarks will not be without value, should they impress you with the great responsibility that will rest upon you, should it ever be your duty to command in the armies of your country.

The Trinity River Survey.

BY

Maj. T. H. JACKSON,
Corps of Engineers.

The River and Harbor Act approved July 25, 1912, provided for an allotment of $100,000 for an accurate instrumental survey of the Trinity River below Dallas, Tex., including a line of precise levels.

The control of this survey has been completed, and it is the object of this article to indicate the methods considered for horizontal control, and the method adopted—primary traverse.

There is no question of the superiority of control by triangulation, but, as is usually the case with the best, it is the most expensive. It is desired here to bring out the fact that for difficult river surveys, such as the Engineer Department is being constantly called upon to make, horizontal control by primary traverse is sufficiently accurate for the purpose, and that it has the great advantages over triangulation of much less cost and considerably less expenditure of time. The primary traverse line of this survey, connecting, as it did, two triangulation stations approximately 394.5 miles apart and in adjusted systems, affords a good example of the accuracy that can be expected.

For the reason that the control work of this survey was made under most adverse conditions, the organization and operation of the parties, with the cost of the work, are considered to have particular value to those who may have to make estimates for similar work.

DESCRIPTION.

Dallas, at about Mile 510, is at the head of the present project for the improvement of navigation. Throughout this entire stretch, the river is a typical alluvial stream, flowing through a valley varying from 3 to 8 miles in width, and about 30 to 80 feet below the adjacent high ground.

The river flows through this valley in a channel with banks from 20 to 50 feet in height, the low banks being found in the upper and lower 100 miles.

The river overflows its banks at all points during high flood stages, the highest banks being covered to a depth of about 4 feet in floods like those of 1908, 1913, and 1914. It is estimated that the capacity of the river throughout may be considered as approximately 20 per cent of its flow in floods like those of 1908, 1913, and 1914.

There is no pronounced high or low water season, and the river may be at an extreme low stage during any month in the year. A low water flow of less than 15 cubic feet per second at Dallas, and about 50 cubic feet per second 300 miles below, may be expected annually for months at a time and at any season of the year.

The troughs of the valley are located very near the high ground, and are from 6 to 15 feet below the banks of the river.

The valley, as well as most of the high ground adjacent, is covered with timber, which in the bottoms is virgin hardwood growth. Probably 10 per cent of the land in the lower 450 miles is under cultivation.

About 50 per cent of the river is over 15 miles from a railroad. No wagon roads run parallel to the river, either on the high ground or on the bottoms, and there are few cross roads. All roads are inferior dirt roads, which are almost impassable in wet weather. Land transportation is therefore difficult and expensive.

The statement made above regarding the low water flow, its amount and duration, shows the difficulties to be expected in the case of water transportation. Only the lightest draft quarter-boats can be used in the upper 250 miles during low water, and these are operated only with the greatest difficulty, owing to the limited flow, drift, snags, etc. The conditions were so bad at one time during the period of the survey that the quarter-boats had to be abandoned, and a portion of the work of the primary traverse and precise level parties carried on using land transportation.

CONTROL.

The writer has had considerable experience in connection with river surveys where the control work was carried on in connection with the topographic work, or where it was not completed in time to be checked and plotted for the use of the topographic parties prior to these parties taking the field. The results have always

been unsatisfactory, causing in many cases the replotting of much data or expensive delays awaiting control data. This has been particularly true in the case of large surveys, or surveys where a good control was being obtained. The topographic parties on a large, important survey should not be expected to provide their own control. That work should be done by more experienced instrument men, with superior instruments, and under direct control of the assistant in charge of the survey; it is a case of special work requiring special treatment.

In this survey it was decided to complete the control work, and to plot the control line on the field sheets, which would be turned over to the topographic parties to fill in the topography. Those parties are concerned then, only with getting topographic data, and are relieved of all the responsibility of control work.

The act referred to above called for a line of precise levels from Dallas to Galveston Bay, hence the method of vertical control was fixed. It is discussed in this article simply to show how well the method of horizontal control by primary traverse fits in with the vertical control, and to show the total cost.

In deciding on a plan of horizontal control, three methods were considered:

(a) Triangulation;
(b) A traverse line controlled by independent astronomical observations;
(c) Primary traverse.

Triangulation. Owing to the width of the valley and the heavy growth of timber, a preliminary examination soon showed that a triangulation system in the valley itself was practically impossible and that one on the high ground adjacent was prohibitive in cost, for it was estimated that the triangulation system alone would cost at least $35,000. The precise level line would have cost more than it did under the method adopted, and the location and clearing of the control line with its bench marks, etc., would have cost from 25 per cent to 50 per cent more than it cost under the method adopted. It is estimated that the method adopted made a saving of about $30,000 over a control using triangulation.

Traverse Line Controlled by Astronomical Observations. It was thought for some time that a good control could be obtained by means of a traverse line controlled by independent astronomical observations for longitude and latitude at points about 30 to 50 miles apart. There is considerable information available on this

subject, though it was not known to this office. Correspondence with the U. S. Coast and Geodetic Survey, the U. S. Geological Survey, and the Lake Survey, brought out the fact that this method is not sufficiently accurate, due to what is known as station error. The following is a quotation from Lieut. Col. C. S. Riché, Corps of Engineers, in charge of the Lake Survey, in answer to a letter on this subject:

In reply to your letter of the 22nd instant, regarding the question of securing an adequate control of the proposed survey of Trinity River by means of points located by astronomical observations for the determination of longitude and latitude, I take pleasure in furnishing the following information:

In my opinion no adequate control of a survey intended for improvement purposes can be obtained by astronomical determinations of latitudes and longitudes alone. The unavoidable errors of positions thus determined are too large to afford checks on distances and courses obtained by triangulation, chaining, or even stadia. An approximate check to detect gross errors could be obtained by observations of azimuth along the course of the work. The agreement of an observed azimuth with that computed geodetically from the nearest other observed azimuth would be a proof of the accuracy of the angle measuring and show freedom from large errors in measuring or mistakes in computation.

If the survey is made to depend on triangulation, the agreement of measured lengths of bases with those computed from other bases through the intervening triangles affords the very best kind of a check on the accuracy of the work. Astronomically determined positions could add nothing to the accuracy of such a survey, although they might be of use in fixing the approximate position of the point of origin. The latitude of a point can be determined by the zenith telescope, as far as the accuracy of the observation is concerned, with a probable error of Plus or Minus 0.10 second, or about 10 feet. The probable error of observation of the best Lake Survey determination of difference of longitude (Detroit, Mich., from Cambridge, Mass., by telegraph) is Plus or Minus 0.031 second, and this depends on several nights' work with interchange of observers to eliminate personal equation.

The errors of observation in latitude and longitude are very small, however, as compared with what is known as the station error, due to deflection of the plumb line, caused by variations in density of the earth's strata in vicinity of the station. The probable error in a latitude due to station error is Plus or Minus 3.0 seconds. A distance between stations derived from the difference of two observed latitudes would have a probable error of Plus or Minus $3''.0\sqrt{2}$, or Plus or Minus 4.2 seconds, equivalent to about 420 feet. The maximum error might be three times as great as the probable error. Some station errors in latitude and longitude in the lake region are shown in Report of the Chief of Engineers

for 1902, pages 2900-1. The Columns A-G in tables, astronomic minus geodetic, are the station errors; the greatest error in latitude is Plus 11.32 seconds, and in longitude Minus 14.01 seconds. The astronomic longitudes in this survey were determined telegraphically. The errors, of course, would be far greater with longitudes determined by chronometer expeditions or from moon culminations or occultations of stars.

Accurate observations of latitude and longitude, particularly the latter, require considerable previous practice on the part of an observer; and, as far as determination of distances accurately from these data is concerned, the highest skill of the observer is nullified by the station error.

In only two instances on the Lake Survey have astronomical observations been relied upon in determining the locations of points. This method was used along two stretches of the Lake Michigan shore; on the west side from Milwaukee to Sturgeon Bay; and on the east side from St. Joseph to Point Betsie. The positions of the ends of the stretches, geodetic longitudes and latitudes by triangulation, were known and the positions of intermediate points determined from observed latitudes with zenith telescope and observed azimuths at the stations. The stations, ten to fifteen miles apart, were intervisible and at each station the horizontal angle was read between the back and forward station. The differences of observed latitudes and the azimuths of lines gave the means of determining the differences of longitude between the stations. These stretches are nearly north and south, a condition favorable for the application of the method, but the results did not prove altogether satisfactory and this work has been a constant source of trouble in the chart construction of that region.

In the case of a survey of a stretch running nearly east and west, the difference in observed azimuths at the two ends of a line can be used to obtain the length of line between the points. The difference in longitude multiplied by the sine of latitude is equal to the difference in azimuth at the ends of line, or to the change of azimuth. The result here, however, also involves station error. This method has never been used on the Lake Survey. It has been exploited by the Coast and Geodetic Survey, but to what extent if any it has been practically used is not known. It would seem only expedient to use it under very exceptional circumstances where other methods could not be used economically or where great accuracy was not required.

For further information on this point, the reader is invited to consult Report of the Chief of Engineers, U. S. Army, 1902, pages 2898-2902, and publications of the U. S. Coast and Geodetic Survey.

This method is dwelt upon here to such length, principally to prevent others from considering it as a possible method of control for a survey where triangulation is not practicable.

Primary Traverse. This method is identical in general with the ordinary traverse, except that greater care is taken and higher grade instruments are used. The distances are measured by a standardized tape, and the angles in the traverse measured by a good transit, reading to 20 seconds. Astronomical azimuths are observed every 5 to 7 miles.

This method is used very extensively by the U. S. Geological Survey, and was recommended by both that Survey and the U. S. Coast and Geodetic Survey for the Trinity River Survey in case triangulation was found impractical.

Full instructions governing "Primary Traverse" work, as carried on by the U. S. Geological Survey, are given in "Instructions to Topographers of the United States Geological Survey," published by that Bureau.

Considering the results previously obtained by the U. S. Geological Survey, it was decided that this was the only method of control that promised results with reasonable cost, and it was adopted.

FIELD OPERATIONS.

In order that the best results might be secured, it was decided to carry on the control work using three parties, viz: a locating, a primary traverse, and a precise level party. Each party was under its own chief of party, and all three parties were under the supervision of the junior engineer in charge of the survey. That plenty of time might be allowed the chief of the locating party to cover thoroughly the ground, and to make the best selection of line, his party was placed in the field well in advance of the other parties. The last two parties started at approximately the same time, and soon combined as one party for transportation and subsistence purposes.

Locating Party. This party was organized at Lock and Dam No. 7 (Mile 463), and began field work on November 13, 1912. It was provided with two small boats, 16 by 30 feet, with loaded draft of 11 inches. These boats were used as mess boats, and to carry supplies. The party occupied tents as quarters.

The composition of the party was as follows:

1 chief of party;
1 foreman (in charge of moving boats);
1 instrument man;
2 rodmen (1 placing monuments);
1 cook and 1 assistant;
14 laborers.

This party was charged with the locating and clearing of the control line with the view of selecting a line that would serve to the greatest extent practicable the needs of the topographic work. This necessitated its being as near as possible to the river bank, for most of the topographic work is in or near the river channel. However, on account of the extreme crookedness of the river, and the desire to keep the line as short as possible, it was impracticable to keep near the river bank at all points. Further, it was desired to keep the line on the left bank of the river throughout, thus avoiding crossings which increase the difficulties of the traverse and level work.

It will be seen from the above that much depended on the good judgment of the chief of this party.

All turning points were marked by *bois d'arc* hubs in such a manner as to last the period of the survey. At intervals of about 1 mile there were placed two concrete monuments, 12″ by 12″ by 4′ 0″, in each of which was set a 3-inch galvanized iron pipe 4 feet long with brass cap marked U. S. E. D. Elev.____ B. M. No. _____. Four hundred and forty-three of these monuments were placed. These monuments were placed in pairs, marking the ends of a course, so that an azimuth might be obtained at any bench mark. All monuments served both as bench marks and as reference points. At intervals of about 5 miles, a glass insulator was placed inside and 2 inches below the top of the galvanized iron pipe. The cap on this pipe was the same as others, except that it could be screwed off and insulator point used as precise bench mark. One hundred and ten of these monuments with glass insulators were placed. Sand and gravel for bench marks were secured along the river.

It is worthy of note that a glass insulator, used as described above, is considered a good level bench mark, because its rounded top permits the bottom of the rod to assume only one elevation and because the mark can not be changed in elevation. It is either broken, or it is the same elevation at which it was placed. No kind of a metal plug has this advantage: it may be driven down, in which case its change of elevation is not evident.

The operation of the locating party was about as follows: the chief of party carried on a reconnaissance ahead, locating the line. The line, once located, was cleared out; its length and bearing were determined by the instrument man, and monuments were placed. The instrument work was almost identical with that of the primary traverse party, though much less accuracy was attempted.

It is possible that a justified saving could have been made by eliminating all instrumental work; still, in view of the extreme length of the line, and of the fact that there would be no chance to check it at intermediate points, it was considered that the results obtained by this party would serve as a good check on a possible large error of the traverse party, such as the dropping of a part of a tape length, etc.

This party located, cleared, and placed monuments on 351 miles of line.

The cost of this party was as follows:

Wages field party	$4,098.24
Placing monuments (553)	805.16
Material in monuments	2,461.29
Subsistence	1,892.67
Equipment[1]	530.29
Boats[2]	481.58
Hauling[3]	404.79
Traveling expenses	99.82
Superintendence	896.55
Total	$11,670.39

From the standpoint of economy in connection with the topographic work to follow, the locating of this line was one of the most important features of the control work. The work falling on this party was very hard and disagreeable, and great credit is due to Mr. C. A. Ridenour, Junior Engineer, for the successful manner in which it was done.

Primary Traverse Party. This party was under the charge of Mr. E. L. McNair, Topographer, whose services were obtained from the U. S. Geological Survey by transfer.

The organization of the party was as follows:

1 chief of party;
2 chainmen;
2 axemen;
2 boatmen;
1 cook.

[1]Includes only 50 per cent of value of certain equipment that was later turned over to topographic parties; a transit belonging to the office was used and no charge made.

[2]Includes 50 per cent of cost of boats: balance charged to topographic survey.

[3]This party was furnished a team for hauling supplies. This item includes upkeep of team, and hire of additional teams when needed; first cost of team, etc., $597.50, was charged to equipment.

Starting on December 5, 1912, at Dallas, Tex., from the U. S. Geological Survey triangulation station Caruth, longitude 96° 46′ 27.42″, latitude 32° 51′ 38.25″ (about 5 miles north of Dallas), this party ran a line along the Houston and Texas Central Railroad to Wilmer; thence along highways to Lock and Dam No. 4 (Mile 482½); thence along highways via Lock and Dam No. 6 to Lock and Dam No. 7 (Mile 463), where the topographic survey was to begin. This line to Lock and Dam No. 7 was approximately 43 miles in length; it was run in fourteen days.

In running the line to Lock and Dam No. 7, the party was subsisted in towns along the way, and at Locks and Dams Nos. 4, 6 and 7. After leaving Lock and Dam No. 7, the party was quartered in tents and had a mess boat similar to that of the locating party.

This party was overtaken at Lock and Dam No. 7 by the precise level party, and the two parties were combined for transportation and subsistence.

The instructions under which this party worked were as follows:

All angles were to be measured with transits reading to twenty seconds. The methods of reading were to be the same as those used by the United States Geological Survey. (See page 78 of "Topographic Instructions of the United States Geological Survey.") Distances along the traverse line were to be measured both forward and backward, with a limiting error of 1:10,000.

This party ran 394.5 miles of line in 1,930 courses. It made sixty-six observations for azimuth. Observations for azimuth were made on Polaris at any hour.

The line was closed on June 28, 1913, on the U. S. Coast and Geodetic Survey triangulation station Wiggins No. 2. This station is located on the shores of Turtle Bay, an arm of East Galveston Bay. Triangulation Station Wiggins No. 2 is in a system of the Coast and Geodetic Survey along the Gulf coast. Triangulation Station Caruth is in the system of the U. S. Geological Survey, which system has been tied to the Coast and Geodetic triangulation system along the 98th meridian. This latter system is tied to the Gulf coast system at Corpus Christi Bay.

The coordinates of the Triangulation Station Wiggins No. 2 are given as 94° 42′ 10.728″ longitude, 29° 49′ 23.682″ latitude. From the computations of the primary traverse line, they were found to be 94° 42′ 08.164″ longitude, 29° 49′ 21.608″ latitude, giving a difference of 02.564″ longitude, and 02.074″ latitude, or 225.85 feet and 209.51 feet, respectively.

As stated previously, the allowable error in an individual course was 1:10,000. It will be seen from the above that the error of closure was approximately 1:6,761.

The weather conditions were so bad, and so much time was lost in moving camp, that though the party was in the field two hundred and two days, work was possible on the line but ninety-five and one-half days.

The cost of the field work of this traverse line was as follows:

Wages field party	$2,687.58
Subsistence	905.40
Traveling expenses	291.38
Equipment[1]	312.64
Boats[2]	271.94
Miscellaneous	162.57
Instruments	410.50
Superintendence	1,386.22
Total	$6,428.23

Precise Level Party. This party, under the charge of Mr. W. P. Stewart, Junior Engineer, was organized as follows:

1 observer;
1 recorder;
2 rodmen;
1 sun-umbrella man;
1 wind-shield man.

It started on December 1, 1912, from the U. S. Coast and Geodetic Survey Bench Mark A-2 at the Court House, Dallas, Tex., (Elevation 433.969 feet M. S. L), and ran a line along the most direct practicable route to Lock and Dam No. 2 (Mile 489½), thence along the west bank of the river via Lock and Dam No. 4, thence along highways via Locks and Dams Nos. 4 and 6, to Lock and Dam No. 7 (Mile 463), where the survey was to begin. This line was 38½ miles in length, and was run in fourteen days.

This party was quartered and subsisted in the same manner as the primary traverse party, to which it was attached later for convenience of transportation and subsistence.

The methods employed by this party were in general those used by the Mississippi River Commission. After the combining of this party with the primary traverse party, it was necessary for it to

[1]There is charged to this work, only 50 per cent of the cost of equipment later turned over to the topographic parties.

[2]Fifty per cent of cost of boats is charged to this work; balance to topographic survey.

increase its rate, and in order to do this the method of observing was changed, and back sights were always read first, both on direct and reverse lines. The instrument used was Buff & Berger precise level No. 2768. The rods were of the Molitor design, the reading of the bottom of each one being zero. Foot plates were used for turning points for about 10 per cent of the line, and iron pins for the remaining distance.

At Wallisville, about 2½ miles from triangulation station Wiggins No. 2, this party left the control line, and ran a line around Turtle Bay through Anahuac, to a bench mark near Round Point, Tex. The work on the line was completed July 3, 1913. The elevation of the bench mark at Round Point was determined by the U. S. Engineer Office, Galveston, Tex., by simultaneous tide gage readings at Round Point, Red Fish North, and Red Fish South, in connection with a general determination of levels of various points in or near Galveston Bay. (See Report of the Chief of Engineers, U. S. Army, 1891, pp. 1879-1890.)

The elevations of this bench as determined by the Galveston office and by this precise level line were:

U. S. Engineer Office, Galveston	18.416 ft. M. S. L.
Precise level	20.404 ft. M. S. L.

From a comparison of the above elevations it appeared that there was a large error in the precise level line, and to determine its amount, an auxiliary line was run from Liberty, Tex. (Mile 41), to Houston, Tex., where a connection was made with the U. S. Coast and Geodetic Survey Bench Marks J-8 (Elevation 46.516 ft. M. S. L.). This line was run July 7 to 24, 1913. The elevations of this bench as given and as determined by the Dallas office were:

Dallas Office	46.516 ft. M. S. L.
U. S. Coast and Geodetic Survey	45.006 ft. M. S. L.
Error	1.510 ft.

Special difficulties were encountered in running this line, on account of the heat. To verify the line it was re-run November 18 to December 12, 1913, with the following results:

Dallas Office	46.685 ft. M. S. L.
U. S. Coast and Geodetic Survey	45.006 ft. M. S. L.
Error	1.679 ft.

The error 1.51 having been previously distributed uniformly in the distance Dallas to Liberty, no change was made following the re-running of the Liberty-Houston line. The same unit correction was applied to the distance Liberty to Anahuac, and the corrected elevation of the bench mark near Round Point was found to be 18.904 feet, or a difference of 0.487 foot from that obtained by the U. S. Engineer Office, Galveston, Tex., by simultaneous tidal observations.

The precise level line from Dallas to Round Point is 398.2 miles in length. In connection with this line there were run several short lines to locks, etc., with a total length of 14.2 miles.

The total length of precise level line run between Dallas and Round Point was 412.4 miles. About 80 per cent of it was run along the river bank and about 20 per cent of it along roads and across open country. There were numerous bluffs along the line, and many creeks, but on the whole the conditions were favorable, and comparatively little trouble was experienced in securing fair closures. Nine per cent of the line was re-run.

The party was in the field on this line two hundred and fifteen days. Field work was done on one hundred and thirty-two and one-half days.

The cost of the field work on the precise level line was:

Wages, field party	$2,747.10
Subsistence, field party	945.44
Traveling expenses, field party	76.85
Equipment[1]	351.48
Boats, etc.[2]	217.94
Superintendence	1,536.23
Total (412.4 miles)	$5,875.04
Cost per mile run	14.246

The auxiliary line, Liberty to Houston, was 41.5 miles in length; it was run along the highways and the railroad.

[1]There was charged to this work, only 50 per cent of the cost of such equipment as was later turned over to the topographic parties. The precise level and rods were borrowed from the Lake Survey.

[2]Fifty per cent of cost of boats is charged to this work; balance to topographic party.

The cost of the field work of the precise level line from Liberty to Houston was:

Wages, field party	$348.00
Travel	72.25
Subsistence	113.50
Total	$533.75
Cost per mile run (41.5 miles)	12.86

The cost of re-running this line in November and December, 1913, is not given, as a special party was organized for that work.

COMPUTATIONS.

The computing of the primary traverse line was done in the Dallas office. Work was begun on May 10, 1913, and completed on August 25, 1913.

As stated above, the error in closure was 225.85 feet in longitude, and 209.51 feet in latitude. The line was adjusted for these errors, the errors being distributed equally throughout.

The cost of the computations and adjustments was:

Wages	$1,248.83
Superintendence	75.00
Miscellaneous	19.88
Total	$1,343.71
Average per mile	3.26

The formula used in these computations is that of the U. S. Coast and Geodetic Survey, i. e.—

$$-\triangle\phi = S \cos \alpha \; B;$$
$$\triangle\lambda = S \sin \alpha \; A' \sec \phi';$$
$$-\triangle\alpha = \triangle\lambda \sin \phi'.$$

When

S=Distance in meters between stations;
ϕ=known latitude;
λ=known longitude;
ϕ'=unknown latitude;
λ'=unknown longitude;
$\triangle\phi$=difference in latitude;
$\triangle\lambda$=difference in longitude;
$\triangle\alpha$=difference in azimuth;
A′ and B=L. M. Z. coefficients.

The use of the U. S. Coast and Geodetic Survey formula, rather than that of the U. S. Geological Survey, was decided upon for the following reasons:

(*a*) With the U. S. Coast and Geodetic Survey formula, each course must be computed separately, and therefore the labor involved is more than that with the U. S. Geological Survey method which computes by combining several courses in one computation. However, the U. S. Geological Survey method introduces a cumulative error, which is undesirable, and this error becomes serious when the combined courses exceed approximately 4,000 feet in length. For accurate work, therefore, the U. S. Coast and Geodetic formula is better.

(*b*) The computer was familiar with the use of the U. S. Coast and Geodetic Survey formula.

(*c*) To derive any benefit from the use of the U. S. Geological Survey method of computation, a special computing machine was necessary, which would cost more than the labor it might save.

The following tables were used, and are recommended for use by those making similar computations:

Logarithms of Sines and Tangents for Every Second; by Robert Shortrede. Published by Charles and Edwin Layton, London, England. $10.50.

Logarithmic Tables of Numbers and Trigonometrical Functions; by Baron von Vega. Published by Lemcke & Buechner, 30-32 West 27th St., New York, N. Y. $1.88.

Formulæ and Tables for the Computation of Geodetic Positions, Department of Commerce, U. S. Coast and Geodetic Survey.

The computations for the precise level line were made in the Dallas office. Work was begun July 24, 1913, and completed on August 24, 1913.

The cost of the computations and adjustments was:

Wages of computers, etc	$250.00
Superintendence	75.00
Total	$325.00
Average cost per mile	0.79

RÉSUMÉ OF COST.

As stated before, lines were run from Dallas or vicinity to Lock and Dam No. 7 (Mile 463), where the survey work was to begin. The conditions under which these 40 odd miles were run were so

different from those governing on the control line, that the cost is kept separate in the following summation:

Cost of locating control line (351 miles)	$11,670.39
Cost of horizontal control, Dallas to Lock and Dam No. 7 (43.5 miles)	225.35
Cost of horizontal control, Lock and Dam No. 7 to Gulf (351 miles)	7,546.60
Cost of vertical control, Dallas to Lock and Dam No. 7 (38.2 miles)	382.10
Cost of vertical control, Lock and Dam No. 7 to Gulf and to Houston (412.4 miles)	6,351.69
Total cost	$26,176.13

CONCLUSIONS.

From the above it will be seen that the results of the primary traverse work were very satisfactory. Having a triangulation station at each end made it possible to obtain a check on the results, and to make an adjustment. The error in this case was so small that no appreciable error would have appeared in the final maps had there been no adjustment.

The precise level work was not satisfactory. While sufficiently accurate for the purposes of the survey, it is regretted that the results were not sufficiently high grade to be received with full credit by the U. S. Coast and Geodetic Survey.

The cost data were kept carefully, and are reliable. No account was taken of the value of the equipment at the end of the work, and since that value was undoubtedly greater than the value of the old equipment used, the cost is slightly higher than it should be, rather than lower. Conditions as to old equipment and equipment on hand at the end of the work are about what would ordinarily obtain in an Engineer district.

There is no question of the superior accuracy of control by triangulation, and it is not the intention to dwell on the primary traverse method as its equal. It is maintained, however, that the triangulation method for river surveys like that of the Trinity River is too expensive for the results desired, and that control by primary traverse is well adapted to such surveys.

An examination of the cost data above will show that the big single item of the cost of the control was the locating of the traverse line. If triangulation had been used for control, this same traverse line would have been necessary. In addition, lines connecting the traverse line with triangulation stations would have been neces-

sary. With triangulation, the instrument work on the traverse line would not have had to be so carefully done, but the saving in cost on account of this would not compare with the extra cost of the additional lines connecting traverse with triangulation stations. So the conclusion that the traverse line would cost more with triangulation than without it is unavoidable.

Of course there was the danger of errors in the primary traverse line, but since the actual work on the line cost but $7,771.95, it could have been re-run at least four times for the estimated cost of the triangulation.

To Capt. Ernest Graves, Corps of Engineers, is due the principal credit for the methods adopted for the horizontal control, and for the computations. Captain Graves also outlined the general scheme of locating and marking the control line.

The field work was under the immediate charge of Mr. T. B. Warden, Junior Engineer, and great credit is due him for the satisfactory results obtained, both in the successful prosecution of the control work, and the completeness of the cost data.

GENERAL REMARKS.

The following are a few general remarks regarding river surveys:

Good Surveys. It is a great mistake to make anything but accurate, well monumented surveys, with maps made with large scale.

There should seldom be any surveys between field reconnaissances and high grade instrumental surveys.

If a waterway is of sufficient importance to justify a project, it is of sufficient importance to have a survey that will settle definitely the possibilities not only of present, but of future, navigation.

A case can be cited where three surveys of a waterway have been made in a period of twenty-two years, and in each case practically no traces have been found of the preceding survey. They were surveys that were supposed to be rather good, too. Too many projects are based on the insufficient data furnished by a poor survey. It would seem advisable to get the survey before the improvement begins.

A survey of an alluvial stream should be made with the view of rendering available, to the greatest extent possible, the data that are now necessary, or will be necessary in the near future in connection with the flood and reclamation problems of the waterway. These latter problems are in general more important than the

navigation problem, and their importance is increasing rapidly. Money spent on the survey of such streams will seldom be lost.

Two hundred to three hundred dollars per mile of river is not too much to spend on the survey of an alluvial stream, the improvement of which for navigation will cost from $25,000 to $50,000 per mile.

The cost of the Trinity River Survey will be about $250.00 per mile. This survey will cover thoroughly the channel and the ground for about 1,500 feet on each bank. For about 1 mile on each bank the ground is covered to an extent sufficient for a consideration of the flood and reclamation problems. An additional $50.00 per mile would permit the covering of the entire valley to an extent desired in a consideration of these affiliated problems.

Monuments. The placing of other than the most permanent bench marks is a mistake which can be appreciated only by those who have tried to find traces of bench marks, etc., located in connection with a previous survey.

It is a waste of time and money to make a survey that is to have any future use, and tie it to nails in trees, etc. In the case of alluvial streams, bench marks of triangulation points that are buried have little more value than nails in trees. Only large concrete monuments (one, or better two, to the mile, and extending above the surface of the ground) will probably ever be found years after the survey is made.

Maps. In making maps there is a tendency to get an artistic, rather than a useful article. Too many maps are made that for accurate work are of little use to anyone, except in connection with field notes.

Each sheet of a map should be complete in itself, and independent of the field notes. This can be done by placing upon the sheet all data indicating the position, the elevation, and the azimuth of all permanent marks. If this is done it is of little importance should the field notes be destroyed. It is considered especially important that the azimuth between adjacent marks should be given, for in the case of future work involving the use of such data there will be a great saving of time and labor.

Scale. The scale of a good map should be large, irrespective of the area covered. Too many maps are drawn to so small a scale that few, if any, details can be shown. Again, if the scale is large, the errors or scantness of a poor survey are more readily recognized.

Hand-Grenades in the Russo-Japanese War.*

[Translated from the *Voenni Sbornik*, January, 1910. Communicated by the General Staff, War Office.]

A long-forgotten weapon, which for years had rusted uselessly in our arsenals, was reintroduced in the recent war. This was the hand-grenade. As a result of the peculiar character of the operations grenades were much used, and under certain circumstances, especially at Port Arthur, their employment produced a decisive effect. Scarcely any other weapon was thought of when the enemy was met in deep ditches or had to be driven out of strongly casemated buildings; and, generally speaking, they were of the greatest value in all hand-to-hand fighting.

It might be urged that the reappearance of grenades was only ephemeral, that it was due to the exceptional conditions of the war, and will not be repeated. But this view will not bear examination. Careful study of the war shows that with the development of quick-firing weapons, troops in action involuntarily strive to increase the intensity of their fire and to protect themselves.

The Japanese infantry, which is of the best, carried bags which they sometimes filled with earth in order to protect themselves from fire, and their firing lines often entrenched themselves.

In future we shall seldom see great battles which last only a few hours, like Leuthen and Rossbach, or a single day, like Mars la Tour, Gravelotte, etc. Field fortification is coming to its own. A force which holds a defensive position, strengthens it with all the contrivances of military engineering science. The attacker, in order to crush his enemy, must resort to close combat, and here the hand-grenade comes in. It is indispensable both in fortress and in field warfare, and probably all powers will reintroduce it.

The use of the grenade dates from the 15th century. In the 17th it was a favorite weapon, but since the 18th its popularity has declined. Both French and Russians used it at Sevastopol, as did the British in the Soudan, 1884-1885. In the last war it was first used at Port Arthur, in August, 1904, and from that time it became the principal weapon at close quarters. The Russian grenades were at first improvised from old iron cases or mountain

*Copied from an article written by A. Bortnovski in the *Journal of the United Service Institution of India*, Vol. XXXIX, 1910, pages 531-535.

artillery shells. Later, the brass cases of quick-firing artillery, cut down to a height of 4 inches, were pressed into the service. They were filled with dynamite or gun-cotton, and were fitted with an ordinary Bickford's fuze burning fifteen seconds. The first Japanese grenades were made from old preserve-tins or bamboo tubes filled with a pound of gun-cotton, and with a similar fuze. When first introduced, the fuzes were often cut too long, and the grenades were thrown back by the enemy. It was lighted with ordinary matches, but later on a rough percussion arrangement was improvised by means of a rifle cartridge, which acted as a primer, and steel wire.

Having once realized the value of grenades, both belligerents naturally sought to extend their sphere of action. Wooden mortars were introduced which threw bombs up to a range of 233 yards. These mortar tubes, which were of about 5 inches caliber, were fitted to a wooden stand and were given a constant elevation of 45 degrees. Ranges of from 58 yards to the limit already mentioned were obtained by varying the propelling charge.

The Japanese invented a very powerful grenade by packing a prism of picric acid between two cakes of gun-cotton. The whole was wrapped in paper tied with string and weighed about 1 pound. A Bickford's fuze with a fulminate primer was attached, and was lighted by means of tinder. This bomb was carried in a bag slung round the neck.

The Russians used various high explosives in their grenades, such as "Sampson" blasting powder, "Raskaruk" and "Divine's Explosive." The charge was covered with a coating of pitch, and was fired by means of a Bickford's fuze with fulminate primer. The bodies of old Japanese shrapnel were also used as cases. Each body was packed with two cakes of gun-cotton held in place by a piece of wood, and was fired by means of the usual fuze and primer. A soft-headed match was embedded in the end of the fuze, its head being covered with wadding for safety.

The Russian workshops in Port Arthur could turn out 2,500 grenades in twenty-four hours. In the month of August alone 18,000 grenades were prepared.

The men had great faith in them, and would often drop their rifles so as to have their hands free for throwing.

Grenades were also extensively employed in the field armies. At Mukden the Japanese used a metal cylinder fitted with a wooden handle and with percussion action. Vertical impact was secured by means of a leaden ring round one end of the cylinder. After Mukden a pattern was adopted which had thick walls and no leaden ring; vertical impact being obtained by means of a guide-tape fastened to the handle. The Russian patterns were generally similar in construction to the Japanese, and had much the same percussion arrangement. The weight of both Russian and Japanese grenades was from 1 to 2 pounds.

On both sides the percussion grenades proved to be by far the

best, especially those which were fitted with a safety arrangement. The grenades fired by means of a Bickford's fuze were unsatisfactory, the matches or tinder used to light the fuze often failing owing to damp or other causes. The fuze itself also required careful handling. The percussion grenades were thrown by means of the handle up to a distance of 60 paces. Leaden rings round the grenade flew in fragments such a distance upon impact that they were dangerous to the thrower.

Grenades have a two-fold effect, produced by the force of the gases generated, and by splinters. The large bombs weighing from 3 to 4 pounds, with thick envelopes, had a great effect both from gases and splinters. These were used at Port Arthur. The effect of the smaller grenades used in field warfare was chiefly moral, although those nearest the burst were killed by the gases, and others were severely hurt by splinters. The radius of action of these small grenades was from 20 to 30 paces.

The correspondent of the *Daily Mail*, on the Japanese side at Port Arthur, gives a striking account of the effect of large grenades at 203-Meter Hill. In one instance fifteen to twenty corpses lay in a heap, terribly mangled, and in some instances with the uniforms torn off by the force of the explosion. All this was caused by one dynamite bomb. Further on, the dead lay in hundreds, often in groups of from ten to fifteen mown down by the grenades. According to Osten-Sasken, the effect of a heavy bomb falling in a trench was such that the assault of the attacker was greatly facilitated. Similarly, a few grenades thrown among the assaulting troops may effectively repel them. In either case, troops take some time to recover from the mental shock.

Thus, light grenades of from 1 to 2 pounds, with thin envelopes, are quite sufficient for moral effect and, being comparatively light, field troops may be equipped with them. The heavy grenades of from 3 to 4 pounds, with thick envelopes, are best for material effect, and should be reserved for fortress warfare.

At Port Arthur the Japanese frequently used grenades for the purpose of preparing the assault, especially when the opponent was sheltered in blindages or behind high parapets; thus the Seventh Regiment when assaulting East Pan-Lung-Shan on the 9th-22nd August. The previous day all its attacks had been repulsed, and only a small detachment managed to find cover in an entrenchment which had been evacuated by the Russians. Here the detachment passed the night. The following day, the officers resolved to attack the fort, as retreat was impossible. The engineers who were with the detachment had a few hand-grenades and were sent in advance. They speedily scaled the parapet and threw their grenades. Some blindages and a machine gun were destroyed, and the fort was won after a fierce struggle.

The Japanese soldier, when assaulting, always had a few grenades with him and carried his rifle in his left hand. Thus the assault itself was prepared by throwing hand-grenades with a view to de-

moralizing the enemy. This was well illustrated at 180-Meter Hill, which had two tiers of trenches and was armed with heavy and machine guns. On the 7-20th September the Japanese took the lower tier of trenches, but failed to capture the upper one. The next day, after a thorough artillery preparation, the First Regiment assaulted the upper tier. As soon as the Japanese arrived within bayonet distance they began throwing grenades. The Russians were completely demoralized, and the hill was taken after a brief struggle.

When fighting hand-to-hand both sides used grenades, throwing down their rifles for this purpose, e. g., at 203-Meter Hill.

The Japanese also found them very useful for the purpose of clearing counterscarp galleries after crowning the glacis, as at Ki-kuan-shan on the 18th-31st October.

The Russians made great use of grenades for defending outer ditches which were without flank defense; e. g., at the Shui-hsin redoubts on the 6-19th September. A body of the enemy penetrated to the outer ditch, where our fire could not reach it, but it was subsequently destroyed by means of grenades thrown into the ditch. Grenades were generally used to repel assaults.

On the 10th-23rd November the Japanese attempted to storm the "Trench of Death," which connected forts Ki-kuan-shan and Kobu. Their parallel was within 50 paces of the trench, but could not be worked farther because the ground was steep, and the workers were destroyed by means of grenades thrown from small mortars. As soon as the enemy rushed forward with the bayonet they were met and repulsed by a hail of lead and hand-grenades.

Russian reconnoitering patrols at Port Arthur usually carried grenades.

During the winter halt on the Sha Ho the outposts were equipped with grenades for the purpose of dealing with minor hostile attacks, and when storming the Russian works at Mukden, each Japanese company had from four to ten volunteer grenadiers. These advanced with the parties told off to destroy obstacles and, when the latter were reached, threw their missiles into the Russian works. The Japanese engineers, thus covered, were then able to destroy the obstacles. The task of the Japanese was facilitated by the mistake made by the Russians in placing the obstacles too near the works.

The Japanese cavalry was also armed with grenades and, when pursued by Russian cavalry, threw them under the horses of the latter.

We may, then, conclude that hand-grenades can be used with advantage and under the conditions enumerated below:

1. In the assault, to clear trenches strongly occupied by the defense. The grenades can be thrown when on the move, and, producing both moral and material effect, may effectually prepare the assault and assist the stormers to cross the last 60 to 100 paces separating them from the enemy.
2. In hand-to-hand fighting.
3. To facilitate the destruction of artificial obstacles. Covering

parties throw grenades, while engineers in rear carry out the actual demolition.

4. To drive the defender out of permanent fortifications. To bring up guns is impossible, to destroy the work is difficult. A well-thrown grenade may kill all of its defenders.

5. In the defense of fortified positions. When the stormers advance in waves, they may be brought to a standstill by the use of grenades.

6. In the defense of fortified works, to prevent the approach of parallels and saps beyond a certain point. The attacker, protected by sand bags and parapets, is almost invulnerable to rifle fire, and artillery fire can not be employed.

7. To assist the defense of an outpost line when that of the enemy is near. Reconnoitering patrols may also be equipped with grenades.

8. Cavalry may carry them with advantage.

If the various uses of hand-grenades enumerated above be recognized, it is obvious that troops should be carefully trained in the handling of them during time of peace.

Maj. Gen. Zealous Bates Tower.

BY

Lieut. FREDERICK S. SKINNER,
Corps of Engineers.

Zealous Bates Tower was born in Cohasset, Mass., in 1819. He was appointed to the United States Military Academy from Massachusetts and graduated, July 1, 1841, at the head of his class. He was appointed Second Lieutenant, Corps of Engineers, from the same date. His first year of service was as assistant to the Board of Engineers. In August, 1842, he was recalled to the Military Academy, where he served as Assistant and Principal Assistant Professor of Engineering until August, 1843. From then until 1846 he served as assistant engineer in the construction of the defenses at Hampton Roads, Va. In 1846 he accompanied General Scott's army to Mexico, and was engaged in the siege of Vera Cruz, and the subsequent operations resulting in the capture of the City of Mexico.

For gallant and meritorious conduct in the battle of Cerro Gordo he was brevetted First Lieutenant in April, 1847. A week later he was appointed First Lieutenant, Corps of Engineers. He was brevetted Captain in August, 1847, for gallantry in the battles of Contreras, where he led the storming column, and Churubusco; and Major in September of the same year for gallantry in the battle of Chapultepec, where he was wounded. From 1848 to 1853 he was superintending engineer in the construction of fortifications at Portland, Me., and Portsmouth, N. H., and repairs and surveys of harbor works along the northeast coast. During the next five years he had charge of the defenses of San Francisco, Cal., and built the Customhouse and Appraiser's Store in that city. During the following two years he served on several boards, and on temporary duty in the Treasury Department. He was appointed Captain, Corps of Engineers, in July, 1855.

He became Major, Corps of Engineers, in August, 1861, while serving as Chief Engineer of the defense of Fort Pickens, Fla. He was brevetted Lieutenant Colonel, November 23, 1862, for gallant and meritorious service during the defense of Fort Pickens, and on the same date was appointed Brigadier General, United States Volunteers. He participated in the Northern Virginia campaign, being engaged at Cedar Mountain (where he was brevetted Colonel for gallantry), at Rappahannock Station, Thoroughfare Gap, and at Manassas, where he was severely wounded. Until June, 1864, he was on sick leave of absence on account of his wounds. From June 8 to September 8, 1864, he was Superintendent, United States Military Academy. In the latter part of September he was appointed Chief Engineer of the Defenses of Nashville and took part in the battle of Nashville. He was Inspector General of Fortifications of the Military Division of the Mississippi, 1864-1865, and in the latter year, Chief Engineer of the Military Division of the Tennessee. On March 13, 1865, he was brevetted Brigadier General, United States Army, for gallantry in the battle of Groveton, Va., and on the same date was brevetted Major General, United States Army, and in June of the same year, Major General, United States Volunteers, for gallant and meritorious service in the field during the war.

In November he became Lieutenant Colonel, Corps of Engineers, and served with distinction as a member of engineer and special boards until May, 1867. At that time he was detailed a member of a permanent board of engineers for fortifications and river and harbor obstructions required in the defense of the territory of the United States. He was promoted Colonel, Corps of Engineers, in January, 1874, and was retired at his own request January 10, 1883. After his retirement he lived quietly at his home in Cohasset, Mass., where he died March 20, 1900, at the age of eighty-one.

The memorial to General Tower by the Military Order of Foreign Wars of the United States contains the following:

In the death of Brevet Major General Zealous Bates Tower, United States Army, retired, the country loses another distinguished soldier, and one of the remaining few representatives of the old army. This Commandery mourns its beloved Commander's decease, and remembers with pride the record of his achievements.

The modest simplicity of this gallant soldier and gentleman was striking. He took a just pride in his library, and his mind was a storehouse of knowledge. He wrote with ease and force upon all subjects, both in prose and poetry. General Tower always enjoyed the respect and esteem of the citizens of Cohasset, who will ever proudly recall that this distinguished soldier belonged to them, and that he sleeps still in their midst.

Col. B. Riley, who commanded the Second Brigade, Second Division, Regulars, at Contreras, in his report of the engagement, gave great credit to General Tower, then a lieutenant, calling especial attention to his services.

Colonel Riley says:

I respectfully call the attention of the Brigadier General Commanding to the gallantry and efficiency displayed by Lieutenant Tower of the Engineers, in conducting my brigade from the village of San Geronimo to the enemy's work. Active and indefatigable in his exertions, his services were of the most important character, and merit especial notice.

On account of the wound received while leading his brigade at Manassas, General Tower's brilliant career as a soldier and leader was checked for nearly two years. This undoubtedly prevented him from attaining to the highest military honors. That his skill and ability were recognized by his superiors is shown by the fact that he was placed on duties requiring the highest development of those characteristics. His success in his work amply justified the choice of his superiors.

Rockets and Searchlights.

[Translated from *Kriegstechnische Zeitschrift*, Jahrgang xvi, No. 5, 1913, by Capt. JOHN N. HODGES, Corps of Engineers, U. S. Army.]

The necessity for the employment of searchlights for night operations in war is recognized in all armies, and in the German Army has given occasion for the employment of special searchlight apparatus by the pioneer battalions. It is, however, natural that only a limited number of such technical apparatus can be manufactured while the necessity for a means of illumination exists even in the case of the smaller military units.

The patented military parachute rocket, manufactured by the pyrotechnical factories and shops of Aloys Muller & Sons at Constance (Baden) and Emmishofen (Schweiz), offers a satisfactory substitute for the searchlight and has been thoroughly tested by many of the armies.

The wars of the last decade have shown that the effective method for diminishing losses due to the destructive effects of modern long-range weapons lies in the division of the fighting front into widely separated thin lines.

Naturally the difficulties of troop leading are immensely increased by the above requirement, and it is only through the instrumentality of a complicated and efficient information and signal service that the uniform conduct of an action in the case of widely separated units can be secured. In the case of the decisive engagements of field armies this communication is secured for the most part by telephone and telegraph, supplemented by the other military means of communication such as messengers, etc.

The telephone and telegraph are not, however, always practicable in the case of widely separated detachments, is in the case of bodies of troops shoved far to the front or flank and, furthermore, these means are practically useless in an advance over rough, heavily wooded and impenetrable country. During the daytime there is the possibility of communicating by flags or heliograph, etc.; at night the above methods can not be used.

Besides, omitting from consideration siege operations, the sig-

nificance of night combats for the field armies has increased in importance. Therefore, in order to have a fairly clear view and to avoid the fire of one's own troops, it is necessary to have an illumination of the foreground, if only temporary in character but still sufficient for orienting oneself. Within certain limits this illumination is possible by means of mobile electric searchlights. It is, however, impossible to bring the heavy wagons necessary for the operation of this apparatus promptly to the front under all conditions, especially in the case of bad roads and wooded country. These searchlights have the disadvantage that they can not generally be employed in wooded country; furthermore, the horizontal light rays are reflected by obstacles such as houses, groups of trees, etc., with a dazzling effect, so that objects lying farther behind these obstructions, which it is important to see immediately, can not be observed.

The development of a new military means of lighting that can serve both for signaling and illuminating is desirable.

The owner of the seventy-year-old firm mentioned above has developed this method after expensive tests of over a year in duration. The starting point of these tests was the ordinary rocket, which is driven into the air by means of its expanding gases, the long wooden staff serving as a rudder. These rockets have been employed in war for centuries. The ordinary rockets provided with long wooden staffs are very difficult to use in modern field operations. Among the objections encountered, the following may be named: the transportation is difficult, the firing under certain circumstances is dangerous, and the results obtained are insignificant.

The military parachute rocket, discovered after endless tests, is protected by patents in the different countries, and has the following characteristics:

DISCHARGE.

The rocket is not fastened to a stick, but as shown in Fig. 1, is discharged from a rifle or pistol.

The rocket can be discharged from any place or from any position of the firer. The barrel of the pistol or rifle is so short that fouling from powder gases is improbable.

Fig. 1 shows the correct position (aiming) of the signal gun or signal pistol for discharging the parachute rocket. With too large an angle of elevation insufficient distance horizontally results; with too small an angle the illuminating body comes to earth too quickly, so that its effectiveness is to a considerable extent destroyed. It

is, therefore, important to give attention to the correct position of firing.

DURATION OF BURNING.

The parachute rockets have an average duration of burning of about forty seconds, the parachute to which the illuminating body is attached opening at the instant of the explosion of the rocket.

THE STRENGTH OF ILLUMINATION.

The strength of illumination is quite remarkable and is sufficient to enable one to read writing. The parachute serves as a reflector, and all the rays of light are reflected downward in the form of a cone. The illuminated zone amounts to about 545 yards.

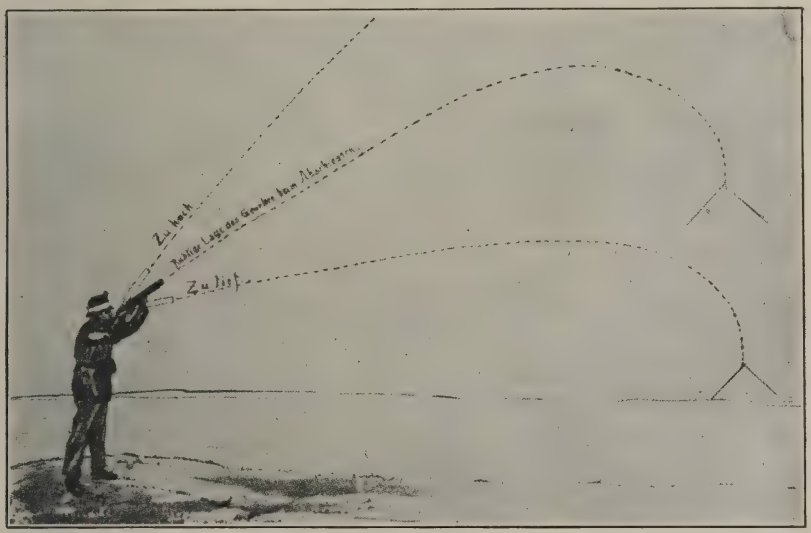


Fig. 1.

LIGHT AND COLOR.

For illuminating work the light is white; for signal work it can be white, green, red, blue, etc. The rockets are also manufactured burning alternately two or three of the above colors. One can also arrange all the desired signals by different arrangements of the colors.

SIGNALS.

In flat country the signal can be seen at a distance of about 31 miles.

WEIGHT.

A complete military parachute rocket weighs about .35 pound, so that a man can easily carry a large number in his knapsack. The equipment necessary for firing these rockets (pistol or rifle)

is somewhat lighter than the service weapon; about 3.3 and 5.5 pounds.

The new military parachute rocket is an important implement that performs inestimable services, not only in hand-to-hand combats, but also in signal and intelligence work, and where these rockets are once provided, they can not be dispensed with because their simplicity and certainty of operation are of great value and work much more advantageously and efficiently than many complicated illuminating apparatus, which, due to their great weight, require transportation by railroad or traction. The Muller rockets can be extensively employed for signaling on the open sea, for life-saving work, for searching for wrecks, and they are also valuable in the high altitudes as emergency signals.

Besides the military parachute rocket mentioned the firm of Aloys Muller manufacture, among others, a particularly effective illuminating body in the form of a cylinder which possesses many advantages as a substitute for a searchlight. A number of these flares can be easily stowed away in a knapsack. These military flares can be manufactured so as to secure any duration of burning desired. The same firm makes metal reflectors for use in connection with the flares, so as to secure concentrated illumination. The flares can be placed with reflector and burned so as to concentrate the rays of light on any desired feature of the landscape. It may also be remarked that this firm manufactures a shrapnel rocket for marking shrapnel shots which, in a manner similar to that employed in the parachute rocket, can be discharged from a rifle or pistol, and which produces a heavy explosion instead of illumination.

Bombs for artillery fire are also made by this firm, which has accumulated much valuable information on the combined uses of signal and life-saving appliances by means of experiments and tests during the past seventy years. For nearly thirty years the Muller rockets were carried exclusively for use as signal lights in case of danger by the Government boats of Switzerland, Baden, Württemburg, Bavaria and Austria.

Highly important innovations for technical use in time of war may be derived from this entirely new discovery: such as searchlights suspended in the air from airships, free balloons and aeroplanes, by means of which night landings from ships, steamboats, etc., are facilitated, as the coast can be rendered as light as day; also, the illumination of bodies of hostile troops can be effected from air craft as desired, and this from any height.

Fig. 2.

Fig. 3.

This discovery should play an important rôle in the future in military art, particularly as applied to aeronautics.

The Muller rockets should be of great service, especially to the pioneer and communications troops in their manifold night operations in the field and also in siege operations.

The excellent operation of the rockets is illustrated in the figures.

Fig. 4.

Fig. 2 shows the operation of a patented military parachute rocket discharged from a signal rifle or pistol, which has brightly illuminated the country for a circumference of 500 to 600 yards, and which offers the opportunity of firing effectively on hostile troops operating over this terrain. In case it is desired to have the terrain illuminated for a longer time, it is necessary to supplement the rocket about to be extinguished by a new one.

Fig. 3 illustrates terrain illuminated by the Muller military flare,

by which a good opportunity is offered of firing effectively with small arms and machine guns from a covered position upon an attacking enemy without being observed.

Fig. 4 shows a free balloon which, by means of a Muller searchlight freely suspended in the air, has brightly illuminated the underlying terrain to a desirably large circumference without the balloon itself being included in the zone of illumination, and for

Fig. 5.

that reason the balloon can neither be seen or heard. This method can be used to illuminate hostile country or to give valuable assistance in making dangerous landings. Two systems of this method of illumination are provided for. In one case the light is released and ignition secured by pressing on an electric contact button. Under these conditions the light floats freely in the air without being in connection with the balloon and casts its strong rays downward for a considerable length of time. In the other case the light

remains in closer communication with the balloon and is connected to it by a line which, by means of a reeling-in and paying-out attachment installed on the balloon carriage, permits the ignition of the illuminating body at any desired height above the ground. The duration of the illuminating body can be adjusted to any desired time, and in both methods of illumination the balloon is perfectly independent of the light, while the latter can be adjusted to any desired depth below the balloon and is connected only by a cable. In the balloon itself is installed the contact button only, so that all danger is effectively removed from the balloon.

Fig. 5 shows an airship from which a freely suspended searchlight has been utilized for the purpose of searching and illuminating the

Fig. 6.

underlying terrain. Both systems of illumination are particularly useful here, as in the case of the free balloon and with an airship illuminating bodies of considerable size can be employed. Also, both systems are entirely separated from the airship itself, so that there is not the slightest danger incurred by the latter.

Fig. 6 shows an aeroplane which has been detached from a body of troops occupying a position on the right of the picture for the purpose of searching the hostile country, and which is already on the point of returning to its detachment. During its return flight of about 2,000 yards the pilot, by pressing on a contact button at definite intervals, allows three Muller searchlights to fall, which illuminate the whole country so that the artillery, which has taken a position on the hill on the right of the picture, has had a good

opportunity to open an effective fire on the illuminated enemy without disclosing to the enemy the position of the batteries. By using the Muller system, an aeroplane can allow any number of these illuminating bodies to fall and the combined light of these bodies furnishes very quickly an excellent lighting system for the batteries. This method can be of the greatest service to a pilot at night in making dangerous landings.

In addition to the above the Muller apparatus is most useful as an extremely visible signal.

The Muller parachute rockets are unquestionably excellent and their use by all troops is urgently recommended. The advance cavalry particularly should not neglect this new invention. Their great utility is recognized in the navy.

These rockets deserve to become the common property of both the army and the navy.

Miscellaneous Notes.

Why Not a Rational Specification for a Wooden Pile?*

BY

Mr. E. P. Goodrich.

The typical specification for a wooden pile, as used by most railroads, municipalities, etc., and as advocated by most producers, reads about as follows:

The pile shall be so straight that a line stretched from center of butt to center of point will not leave the pile more than 1/100th (for example) of the length of the pile.

Several criticisms instantly arise in one's mind as to this specification.

1. It is obviously impossible to stretch a line from center of butt to center of point; and, therefore, a substitute method has to be employed, which latter should logically also be substituted in the specification. Obviously, the pile will naturally lie on a flat surface so that its greatest curvature is horizontal. A line is then stretched from the butt to the point over the center of each; and, by sighting down vertically along the string, the distance between the projection of the string and the side of pile is noted.

2. What relation should exist, if any, between the crookedness of a pile and its length? Obviously, again, the pile must be straight enough to be placed in the pile-driver. It also must not be so crooked that a blow of the hammer will bend it so as to damage it; and it must not be so crooked that its final load will produce

*Contributed by Mr. E. P. Goodrich, Consulting Engineer, on the Staff of the Commissioner of Public Works, Borough of Manhattan, who invites discussion of his proposed specification. This journal will be available for the publication of comments and criticisms by its readers.

a bending moment in the pile so large in amount as to produce stresses exceeding a safe maximum. This load condition and the phenomena during driving are practically identical in nature, except that one is static and the other dynamic. Now the theory of bent columns, or what is the same thing—of eccentrically loaded columns, or of arches, must of necessity apply to piles.

Again, it is obvious that with relation to members of the general size and shape of piles, the length has no effect upon the stresses due to eccentricity of loading at any section—the latter being due only to the amount of the eccentricity and the area of the corresponding section.

3. What should be the maximum permissible crookedness? It would seem that it should depend upon the permissible maximum bending stress for the material composing the pile and of the area of the latter. Most engineers would doubtless concede a relatively high unit stress as permissible for the condition of combined direct stress and bending of a pile. From this, it is seen that when all piles are designed to be uniformly loaded when finally in place, a greater bend is permissible in a pile of large diameter than in a smaller one, because a less unit direct stress will exist in the large pile and a greater one due to the eccentricity of load may be allowed. Such a specification may not make inspection easy and may possibly be considered impracticable for other reasons, but its logical possibility is evident.

Of course, different kinds of wood must have different permissible bends. Also, the designer's load per mile will influence the allowable bend, if the limit placed on the total maximum unit stress is constant.

Assume, for example, a 12-inch round pile, area where bearing on soil commences 100 square inches, designed loading per pile 15 tons or 30,000 pounds. Then the direct stress per square inch is 300 pounds. If a safe total unit stress of combined compression and bending of 2,400 pounds is assumed [the recommendations of the Committee on Wooden Bridges and Trestles of the Am. Ry. Eng. & M. W. Assn., Vol. X, Part 1, page 564, are for long-leaf pine 3,800 ultimate, 1,300 safe stress, 1,300 $\left(1-\frac{l}{60D}\right)$ for long columns, 6,500 ultimate extreme bending, 1,300 safe], then 2,100 pounds may roughly be assumed as the maximum permissible bending stress due to eccentricity of load at the section. Since the moment of inertia is about 1016.164 and the section modulus is 169.364, the maximum permissible eccentricity of stress at the ground level

would be 11.9 inches. Again at a point approximately 8 inches in diameter, where the total direct stress may have been reduced by half, with an area of about 50 square inches, the permissible bending stress may be assumed at 2,250 pounds. The moment of inertia is 196.704, the section modulus 49.176 and the permissible eccentricity 7.4.

Examples might be multiplied; but it would seem evident from the two given that a modification in the type of specification now in use might be advantageous. It seems obvious that the permissible bend should not be measured in terms of the length of the pile, but rather in terms of its diameter at each point along its length. With this idea in view, the following wording with reference to yellow-pine piles is submitted for criticism:

The pile shall be so straight that a line held in contact with any two points selected so as to give the maximum deviation shall not be distant from the surface of the pile at any point more than the diameter of the pile opposite that point.

Break in Sewer Pipe at Washington Filtration Plant.

BY

Capt. C. L. HALL,
Corps of Engineers; Assoc. M. Am. Soc. C. E.

A rather interesting failure occurred at the Washington Filtration Plant on April 28, 1915, of a type which, as far as is known, has not previously been commented upon in the engineering press.

The filtration plant consists of a number of filter beds which are located in adjoining blocks separated by open spaces called courts. These courts are all paved over with 6 inches of concrete paving, the only interruption in the paving being small plots in which trees are grown. The soil underlying the paving is sandy loam, a large part of which was removed and then replaced when the filtration plant was completed. This removal was necessary in order to lay the sewer pipes.

The sewer pipes are used, among other things, for draining the filter beds when it is desired to discharge the water from the beds, not into the filtered water reservoir, but into the city sewer. In order to secure the necessary head, so that the sewer can discharge by gravity, the pipes were laid at a considerable depth.

In court No. 1 there was laid in the same year that the filtration plant was constructed (1905) a 24-inch terra-cotta pipe with bell and spigot joints, caulked with cement mortar. The pipe was 16 feet below the level of the court paving. The soil immediately overlying it was extremely sandy.

The first indication of anything wrong was the almost total disappearance at 2 a. m. on April 28, 1915, of a small tree placed in one of the open plots of the court previously mentioned. The court pavement was immediately taken up, when it was discovered that the surface layer of earth had dropped, leaving a cavity about 15 feet wide, 24 feet long, and 9 feet deep. This sudden drop of a large mass of earth evidently occurred at the same time as the falling of the tree, and completely smashed the pipe for a distance of 16 linear feet.

The cause of this break was doubtless leakage in the joints, which allowed the water to leak through and liquify the sandy material to such an extent that it entered the pipe and was carried along with the water. Beginning with a small leak, the material surrounding the drain was gradually carried through the pipe to the sewer, causing settlement and small breaks in the pipe. The process had probably been going on for years and became apparent only by the dropping of the supporting surface layer of earth. The arching effect of the earth had evidently supported the earth itself, and the arching effect of the concrete pavement had evidently resisted the ordinary highway pressure on the court. It will thus be seen that there had been taken away gradually, without its becoming apparent, a total of 120 cubic yards of earth.

Steps were immediately taken towards repairing the leak by removing the court paving, driving wooden sheet piling around the break and excavating the overlying earth. It was in this way that the complete collapse of the 16 feet of pipe was discovered. The pipe was badly cracked and in an unsafe condition for a considerably greater distance. The broken pipe is being repaired by the laying of new 24-inch terra-cotta pipe entirely surrounded by 6 inches of concrete.

The interesting part of this failure is that, so far as the writer knows, it reveals an entirely novel danger connected with the laying of sewer pipes with leaky joints. Usually the authorities fear that leaky joints will permit sewage to escape and pollute the surrounding soil, but in this case the soil escaped and permitted a dangerous cavity to occur. In view of the fact that the pavement

of the court is constantly used as a highway for the filtration plant vehicles, it is fortunate that the inevitable collapse caused so little damage.

The only injurious effect of the accident has been that the one filter bed drained by the pipe has had to be put out of service pending the repairs above described.

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First Lieutenant, Corps of Engineers, 1863
Brevet Captain, U. S. Army, 1864
Brevet Major, U. S. Army, 1864

SEE PAGE 644

The Diesel Engine.

BY

Capt. JOHN M. WRIGHT,
Corps of Engineers.

INTRODUCTORY.

The internal combustion engine of the Diesel type is a comparative newcomer in the prime mover field, the first successful machine having been placed in operation in 1897 at the plant of the Machinenfabrik-Augsberg-Nurnberg at Augsberg, Germany. As its name indicates, the engine is the invention of Dr. Rudolf Diesel of Munich, and is said to be the outcome of over fifteen years of scientific investigation into thermodynamic problems. The remarkable thermal efficiencies attained by the first engine, far in excess of the best figure reached by any other form of prime mover in use at that time, at once attracted attention; licenses to manufacture under the Diesel patents were sold to firms in Sweden, England, Belgium, Germany and Switzerland, and every effort was put forth to perfect the new machine and introduce it commercially. Many improvements were made, and the use of the engine was gradually extended; but, owing apparently to patent restrictions, the growth was not as great as was warranted by the merits of the machine. The basic patents expired about 1912 and a remarkable expansion in the Diesel engine industry at once followed. The engine is now manufactured in almost every European country of importance, and is being introduced, though slowly, into the United States.

The slow progress of the Diesel engine in this country is readily explained by the fact that, with our abundant supplies of coal and the cheapness of fuel resulting therefrom, the power producers existing previous to its introduction were very satisfactory. In Europe, where coal was neither abundant nor cheap,

a new method of obtaining power was sorely needed; and the Diesel, which promised to supply that need, and has in a great measure supplied it, was eagerly taken up and its development pushed. However, there are indications that the interest of the United States in the Diesel engine is rapidly increasing. Ten or twelve prominent firms have undertaken its manufacture, among them some of our most reliable engine builders. It seems probable that the next ten years will see the installation of many Diesel plants in this country.

It is proposed to examine the Diesel engine under the following heads:

I. Theoretical consideration of the Diesel cycle.
II. Construction of the engine.
III. Fuel.
IV. Operation and performance. Comparison of types.
V. Applications. Comparison with other forms of prime mover.
VI. Probable future development.

I. THEORETICAL CONSIDERATION OF THE DIESEL CYCLE.

As the Diesel cycle is a development of the Otto cycle ordinarily used in internal combustion engines utilizing gasoline, kerosene or gaseous fuel, a brief explanation of the latter cycle may serve to make the former more clear.

The sequence of events in the theoretical Otto cycle may be seen by reference to the accompanying illustration (Fig. 1), and is as follows:

Heat is received along the line *b-c*, the pressure and temperature rising while the volume remains constant. Adiabatic expansion then takes place along the line *c-d*, this being the working period of the cycle. Heat is then rejected along the line *d-a*, the volume again remaining constant while the pressure and temperature decrease. The cycle is completed by adiabatic compression along the line *a-b*.

In the actual engine this cycle may be carried out in either two strokes or four strokes of the piston, giving rise to the well-known division into two-cycle and four-cycle machines. A typical indicator card from a four-cycle engine is attached hereto (Fig. 2), the portion *d-e-f-a* being somewhat exaggerated for the sake of clearness. The events which take place and are shown thereon are the following:

Beginning at the point *e*, the inlet valve being open the outward

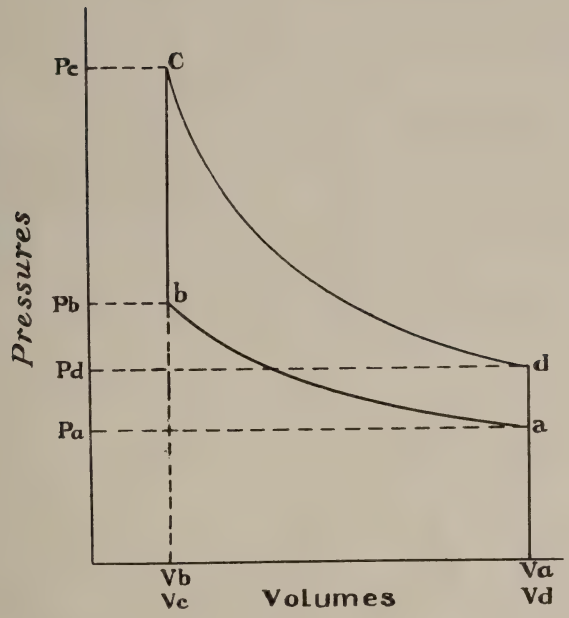

Pc
C
Pb
b
Pd
d
Pa
a
Pressures
Vb
Vc
Volumes
Va
Vd

Fig. 1.

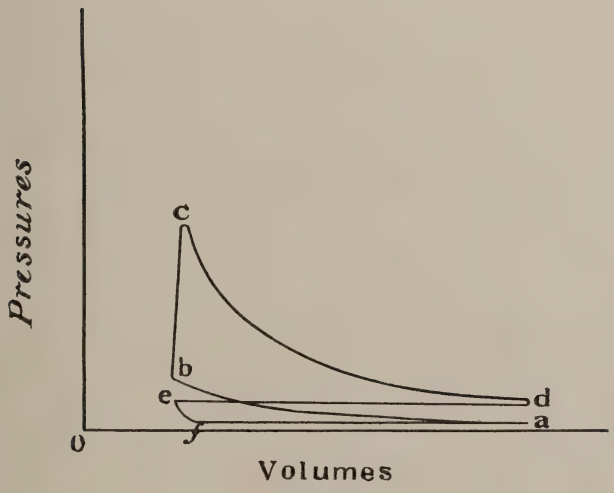

Pressures
C
b
e
d
a
0
Volumes

Fig. 2.

stroke of the piston takes place along the line *e-f-a*, a charge of explosive mixture of fuel and air being drawn into the cylinder. The inlet valve then closes and the mixture is compressed along the line *a-b*, the terminal pressure to be attained at *b* depending on the fuel used. At *b*, the compressed mixture is ignited, usually by electric means, and explodes, giving the great rise of pressure and temperature shown by the line *b-c*. Adiabatic expansion then takes place along the line *c-d*, this forming the power stroke of the engine. Shortly before the end of the power stroke the exhaust valve opens and remains open throughout the exhaust stroke *d-e*, during which time the burned gases are expelled from the cylinder. It will be seen that but one stroke out of the four is a working stroke.

The two-cycle engine performs the same chain of events in a

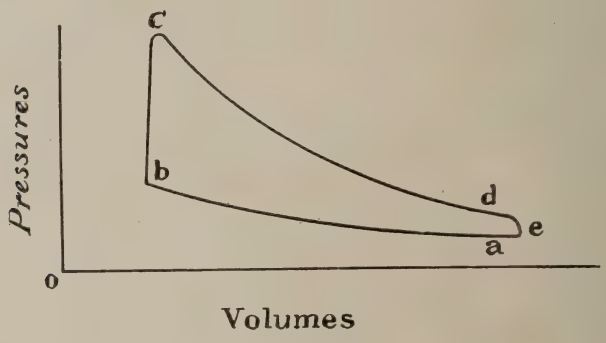


Fig. 3.

slightly different manner. The accompanying card from such an engine (Fig. 3) may be explained as follows:

Let us consider the cylinder to be filled with explosive mixture at the point *a*, the inlet valve having just closed. As before, compression takes place along the line *a-b*, explosion at *b*, and expansion and the working stroke along the line *c-d*. At the point *d*, however, the piston in its outward movement uncovers a ring of exhaust ports in the cylinder wall, and the burned gases pass out through these ports. At the point *e*, there opens the inlet valve situated in the end of the cylinder opposite to that containing the exhaust ports, and a fresh charge enters and moves down the cylinder, driving the burned gases before it. The head of the fresh charge should meet the returning piston at the point *a*, just after the latter has again covered the exhaust ports. The cycle is

thus completed and it will be seen that one stroke in every two is a working stroke.

The theoretical Diesel cycle is shown in Fig. 4. It will be observed to include adiabatic expansion (line *c-d*), rejection of heat at constant volume (line *d-a*) and adiabatic compression (line *a-b*) just as does the Otto cycle. An important difference is to be noted, however, in the fact that the reception of heat along the line *b-c* is not at constant volume, but at constant pressure.

The Diesel cycle may also be completed in either two or four

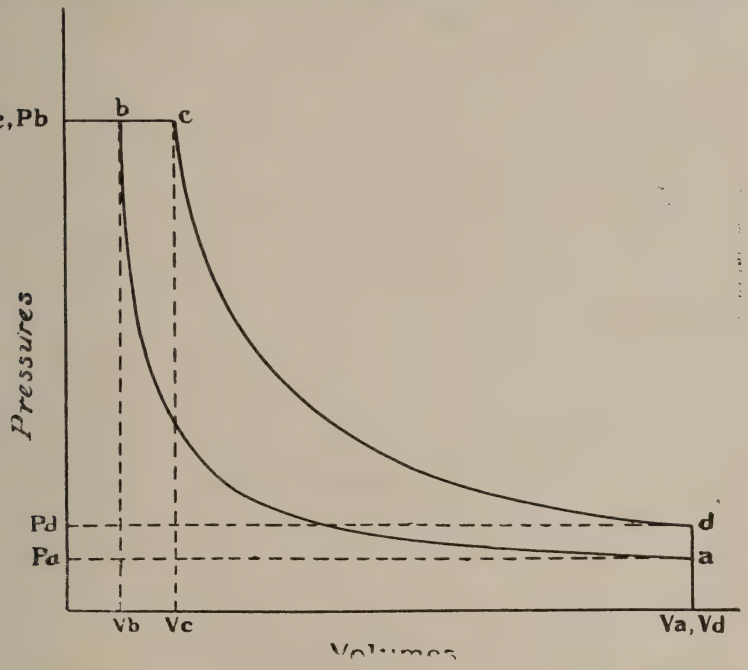


Fig. 4.

strokes, the differences between the two engines being exactly those obtaining between the corresponding Otto engines. An indicator card from a four-cycle Diesel engine is attached (Fig. 5), and the course of events is as follows:

Commencing at the point *e*, as before, the suction stroke takes place, but, instead of an explosive mixture, air, only, is drawn into the cylinder. On the return stroke *a-b*, this air is very highly compressed, the terminal pressure at *b* being about 500 pounds per square inch. Just before the point *b* is reached the fuel valve

opens, and liquid fuel in a very finely divided condition is introduced into the cylinder. The temperature therein has been raised by the high compression to a figure considerably above the igniting point of the fuel, and combustion takes place. The introduction of fuel continues along the line *b-c* until at *c* the fuel valve closes, combustion continues for a very little additional time, and adiabatic expansion along the line *c-d* follows. The events occurring along the line *c-d-e* are exactly those occurring at the corresponding time in the Otto engine. It will be noted that the line *b-c* is not an exact constant pressure line as in the ideal cycle, but that a considerable drop in pressure is observed before the end of combustion.

A comparison of the Diesel cycle, as just outlined, with the Otto cycle, offers the following points of interest:

a. The efficiency of the theoretical Otto cycle is found from the expression

$$\text{Efficiency}=1-\frac{1}{r^{\gamma-1}}$$

in which

r=the compression ratio=$\frac{V_a}{V_b}$ (Fig. 1.)

γ=the coefficient of V in the formula for adiabatic expansion. PV^{γ}=constant. γ=1.41 for air and has smaller values for other gases and mixtures ordinarily employed in gas engines.

The efficiency of the theoretical Diesel cycle is obtained from the equation

$$\text{Efficiency}=1-\left[\left(\frac{1}{r^{\gamma-1}}\right)\left(\frac{R^{\gamma}-1}{\gamma\,(R-1)}\right)\right]$$

in which r and γ have the same meanings as are given above, and

$$R=\text{cut-off ratio}=\frac{V_c}{V_b} \quad \text{(Fig. 4.)}$$

It is clear from the above expressions that the efficiency of both engines varies directly with r and γ, while that of the Diesel also varies inversely with R. On account of the fact that the Otto engine compresses an explosive mixture of gas and air, the permissible terminal pressure must be so low as not to produce a temperature high enough to fire the charge. If this temperature be exceeded, premature explosions are certain to occur. No such

limitation is imposed on the Diesel engine since the gas compressed, being air only, is not explosive. We therefore find that in practice the Diesel compresses to a terminal pressure of 500 pounds per square inch with a corresponding value of r of from 12 to 13, while the Otto engine using gasoline compresses to about 75 pounds with a value of r of about 7. This consideration indicates that the Diesel ought to show much better efficiencies than the Otto.

b. The Diesel engine is not dependent upon some outside means of initiating combustion, such as the electric spark. The mere contact of the fuel with the highly heated air contained in the cylinder is sufficient. Combustion, therefore, does not commence at one point in the cylinder (the spark plug in the case of the Otto

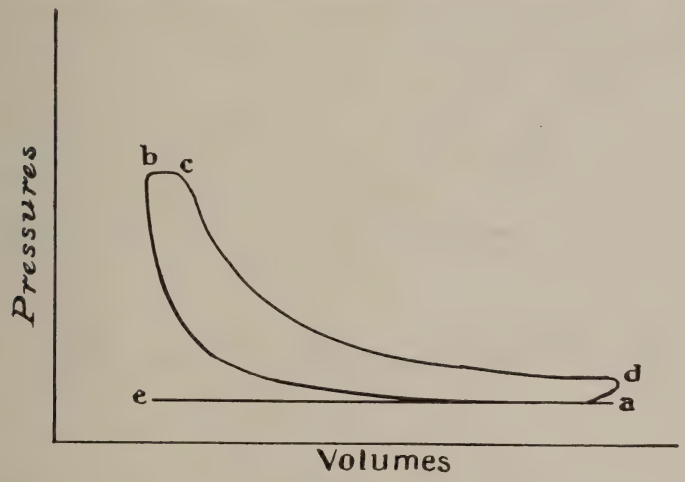


Fig. 5.

engine) and then spread throughout it, but takes place at the same time in all parts of the cylinder in which fuel and air come into contact. The finely divided state in which the fuel is introduced makes it certain that no particle of the combustible can escape this intimate association with the highly compressed air already in the cylinder. Combustion must therefore be almost perfect, and is far more complete than that obtained in the best type of Otto engine.

As a result of this perfection of combustion it is found to be possible to use in the Diesel much leaner mixtures than can be successfully employed in Otto engines. This in itself tends toward fuel economy, and also helps the theoretical efficiency of the cycle

(see expression for efficiency, page 534) by increasing the value of γ and making it more nearly equal to that for pure air. The provision of a large excess of air by the use of lean mixtures is an additional insurance against incomplete combustion.

II. CONSTRUCTION OF THE ENGINE.

It is proposed in this section to take up the simplest variety of Diesel engine—namely, the single cylinder, four-cycle, single acting, vertical type—to explain the method by which it carries out the cycle which has been previously discussed, and to describe its important component parts. The modifications in construction necessary to produce the two-cycle engine will also be pointed out.

The events taking place in the cycle have already been described (page 533), and it is evident that, to perform them, the engine must include the following parts (see Figs. 6, 7, and 8):

a. Frame, cylinder, piston, connecting rod, crank shaft and flywheel—obviously necessary.

b. Valves to perform at the proper time the various functions indicated.

c. Some means of introducing fuel into the cylinder against a pressure of 500 pounds per square inch.

d. A governing device.

e. Means for cooling.

f. Means for lubrication.

g. Means for starting, since the machine is obviously not a self-starter.

These parts will be taken up in the order enumerated.

a. Frame, etc. The frame may be of the ordinary A-type and open, or an enclosed crank case may be provided. Both varieties have been used with the latter in greater favor, both because of ease of construction (since the entire crank case may be cast in a single piece) and of the better solution of lubrication problems which is possible with the use of the crank case as an oil reservoir. The cylinder head is usually constructed separately from the body, and is fastened to it by heavy bolts. The valves are placed in the head. The piston is ordinarily of the trunk type, and must be made of ample length to distribute the side thrust against the cylinder walls. On engines of large size, crossheads are frequently used. The connecting rod, crank shaft, and flywheel require no especial mention, except that in their design and construction regard must be paid to the high pressures used in the engine.

b. Valves. As has been stated, the valves are regularly placed in

A. Bed Plate.
B. Frame.
C. Cylinder.
D. Cylinder Head.
E. Exhaust Pipe.
F. Muffler.
G. Exhaust Gas Outlet.
H. Fuel Storage Tank.
I. Starting Air.
J. Blast Air.
K. Engine Fuel Tanks.
L. Fuel Pump.
M. Air Start Valve.
N. Fuel Valve.
O. Cam Shaft and Gears.
P. Air Inlet.
R. Exhaust Valve.
S. Air Inlet Valve.
T. Air Compressor.
U. Governor Shaft.
V. Flywheel.
W. Governor.

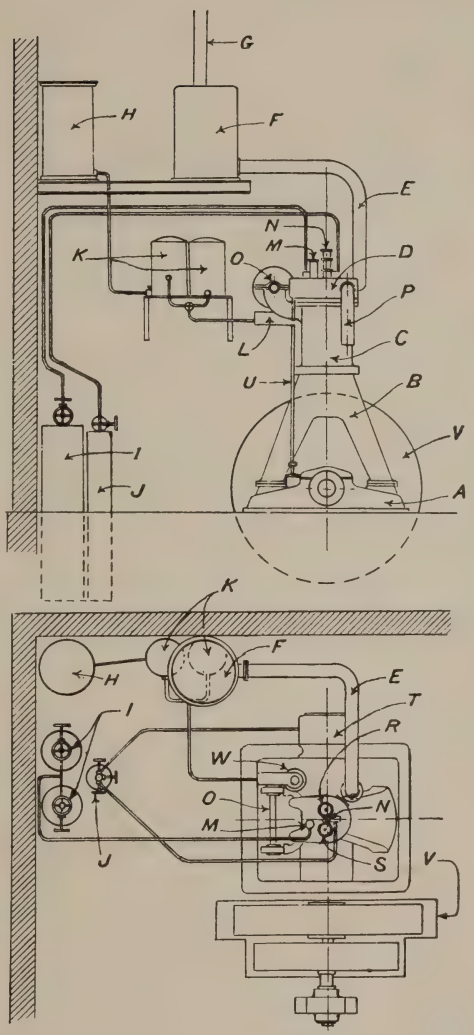


Fig. 6. Section and Plan of Four-cycle, Single-cylinder Diesel Engine Installation.

the cylinder head. They are operated by pivoted levers actuated by a horizontal cam-shaft, though occasionally eccentrics are employed for the purpose instead of cams. The horizontal shaft is usually driven from the main shaft by means of gearing and intermediate shafting. The valves are four in number, and include an air start valve, an exhaust valve (Fig. 9), an air inlet valve and a fuel valve (Figs. 10 to 12). The first three are of the poppet type, and are held on their seats by strong springs. The exhaust valve, of course, opens directly to the exhaust passages, and the starting valve to the air supply used for that purpose; but the air inlet valve communicates with the atmosphere through a cylinder with narrow slots, which serves both to muffle the sound of the intake and to prevent the introduction of dust. The fuel valve is placed in the center of the cylinder head with the starting valve next to it, while the exhaust and inlet valves occupy outside positions.

The fuel valve, which calls for special description, connects with the cylinder through a small tapered opening in which a piece of metal, known as the flame plate, is placed. This plate is perforated with a few small holes. The fuel valve, proper, is set just above the flame plate. Its lower part consists of a conical opening which is closed by means of a long needle extending through the valve and held on its seat by a very heavy spring. The valve is opened by a lever which raises the needle against the pressure of this spring.

The parts just described are common to practically all types of fuel valve, but various designs for the remaining portions are used. The most common form will first be taken up.

Surrounding the needle and just above the tapered seat on which it rests are several metal rings, each perforated with very fine holes. The rings are separated by small spacers and are so set that the holes in successive rings are staggered. The needle works through an interior casing, which is of such a size as to leave an annular space between it and the outside of the valve. This space is over the perforated rings, and is in communication with a compressed air reservoir in which a pressure of from 750 to 1,000 pounds per square inch is maintained. The fuel pump, which must be of special construction to work against the high pressure, delivers fuel to the annular space at a point just above the rings. It is obvious that, whenever the needle lifts, the high pressure air will force fuel through the perforations and through the flame plate into the cylinder. By its passage through the rings the fuel is

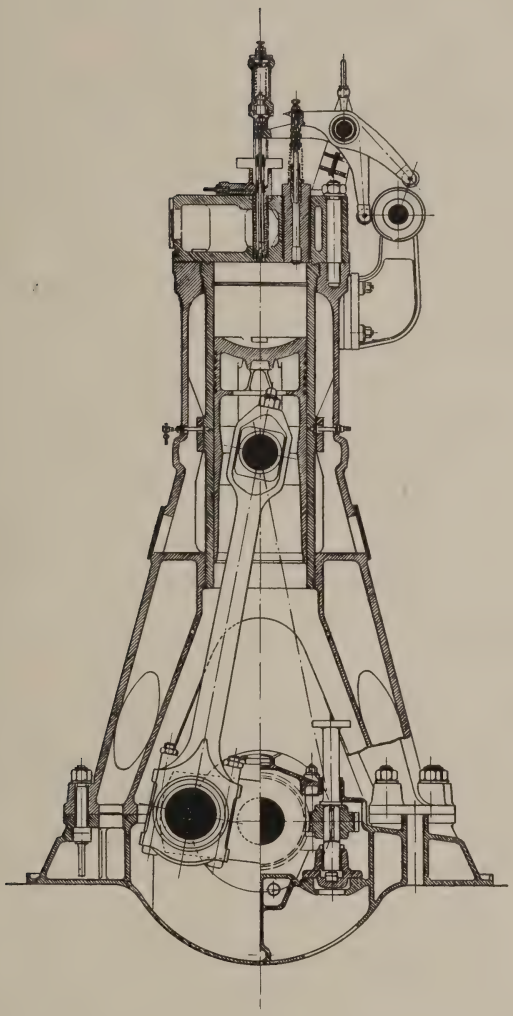


Fig 7. Transverse Section of Four-cycle, Single-cylinder Diesel Engine.

reduced to the form of a fine spray, and enters the cylinder in a condition very favorable for combustion.

Many modifications of the valve above described are in use, probably the best being the Hesselmann atomizer. The action of this valve is shown by the accompanying illustration (Fig. 12), which may be described as follows:

The figure marked 1 shows the valve immediately after injection has taken place, full-blast pressure being exerted from both branches of the Y-shaped passage against the oil in the stem of the Y. The figure numbered 2 shows the state of affairs immediately after the delivery stroke of the fuel pump has occurred, the oil having risen in both branches of the Y. In the third figure injection is in progress. The lift of the needle slightly reduces the pressure on the inside of the valve as the blast air therein rushes into the cylinder. The full-blast pressure, however, is still exerted on the oil in the vertical branch of the Y, and it is forced out through the inclined branch into the interior of the valve. As it leaves the end of the passage it is caught up by the air rushing into the cylinder and is carried with the air in a very finely divided state. It is evident that when both branches of the Y are emptied, no more oil will pass into the interior of the valve, the condition of affairs shown in the first figure then being reached.

This valve has several advantages over the disk type, among them being the following:

1. The valve passages are entirely cleared of oil at each lift of the needle. The disk type of valve always retains some oil between the disks, which causes it to become dirty and to clog rather easily.
2. The exact amount of oil delivered by each stroke of the fuel pump is fed to the cylinder.
3. The atomization is considerably more efficient than is obtained by the disk arrangement.

Engines using valves of the Hesselmann type are said to consume 8 per cent less fuel than those equipped with disk valves.

Some engines must have their fuel valves set horizontally. This requires special design in order to cause the oil to be delivered to the cylinder evenly all around the needle, instead of at its under side only. Others employ two different kinds of oil at the same time, introducing a construction more complicated than is here outlined. Of whatever form it may be, the fuel valve is a part upon which especial care must be exercised both in design and

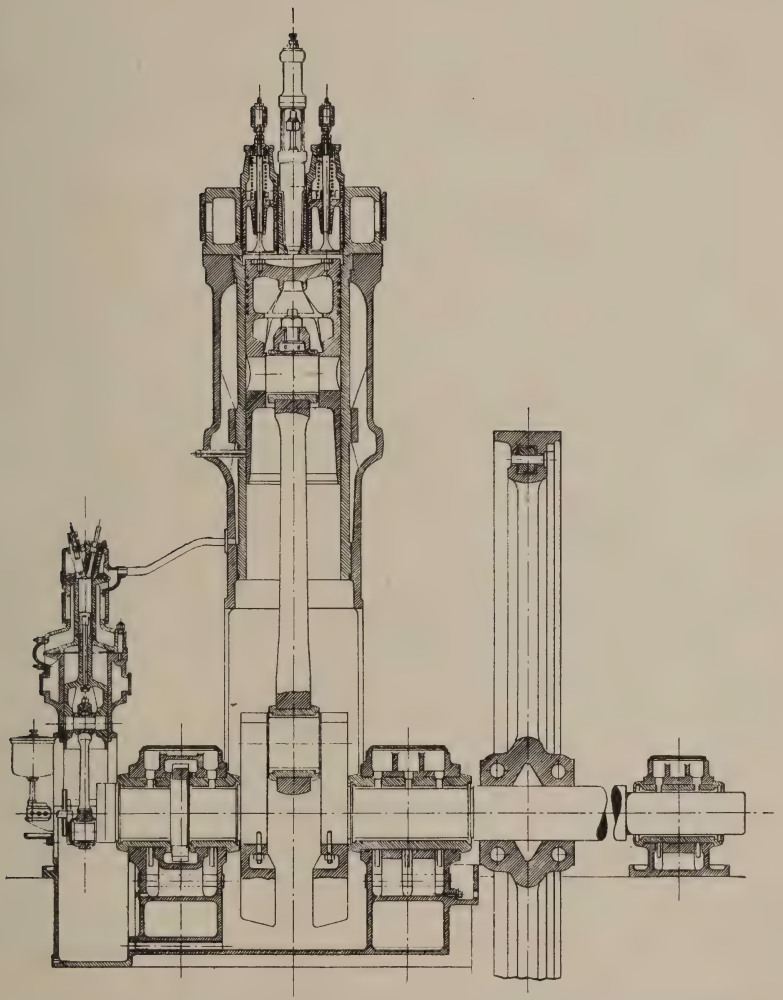


Fig. 8. Longitudinal Section of Four-cycle, Single-cylinder Diesel Engine.

in upkeep. A fault here manifests itself at once, and often in a dangerous manner.

c. Compressor. It is evident from the preceding discussion of fuel valves that the engine, while running, must have a continuous supply of compressed air at a pressure of from 750 to 1,000 pounds per square inch. This very high pressure demands a compressor of special design and exceedingly careful construction, and in many Diesel engines the compressor is the source from which trouble most frequently arises.

Compressors are usually built two or three stage with intercoolers of heavy tubing between the stages. The high pressure piston carries five or six rings, and is fitted into the cylinder with a minimum of clearance. The valves between the stages are usually of the ball or disk type. Each consists of two perforated plates screwed together in such a manner as to leave some distance between the inner portions of the adjoining faces, while raised rings at the circumference of these faces are in close contact. The disk or balls are set in the space thus left between the adjoining faces, and are so arranged as to form a check valve permitting air to pass through the perforations in one direction but not in the other. Extreme lightness and rapidity of action, as well as tightness under heavy pressure, are required of these valves. The cooling of the compressor is usually taken care of by surrounding cylinders and intercoolers with a kind of hood through which water is continuously circulated.

Compressors are best driven by direct connection to the engine crank shaft. Some makers, however, employ chain or link connection between engine and compressor, while one manufacturer, who builds engines for electric power purposes only, uses a separate motor driven compressor. The Reavell compressor, which is widely used on European engines, is driven by an eccentric on the end of the crank shaft, and is a remarkably good type, being very compact and efficient. It is not, however, well suited for high speed work.

It is the ordinary practice to arrange the compressor to deliver air to a storage tank or bottle, and to feed from this tank to the fuel valve, rather than to attempt to deliver air to the fuel valve direct from the compressor. This insures a constant pressure at the valve, and permits uniformity of operation.

d. Governing Device. In connection with the fuel valve it was stated that the fuel is delivered to the valve by means of a pump,

which must be especially designed to work against the high pressure of the blast air. Practically all the schemes for governing Diesel engines depend upon the control by a centrifugal governor of the amount of oil delivered to the valve by this fuel pump. This control is exercised in two different ways:

1. Control of the suction valve of the pump, by which it is held

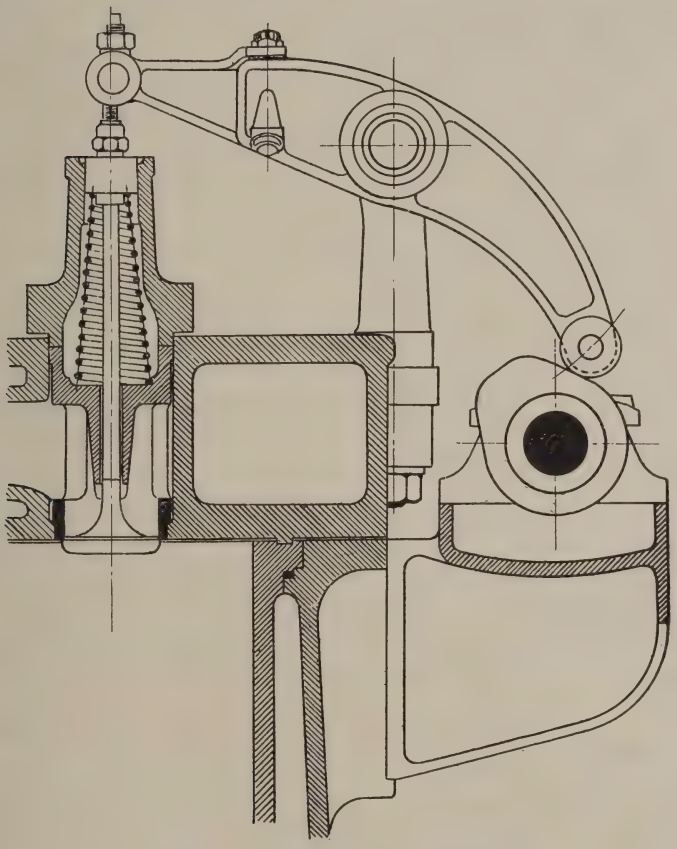


Fig. 9. Exhaust Valve. (Section.)

open during a certain portion of the forcing stroke. This results in the return of a part of the oil contained in the pump barrel to the reservoir from which it came, and the delivery of the remainder to the fuel valve. The less the time of suction valve opening, the greater is the amount of oil delivered. This is the usual method of governing.

2. Alteration of the effective stroke of the pump either by move-

ment of the pump barrel, or by changing the center of motion of the pump plunger. This directly affects the amount of oil delivered to the valve.

Of these two methods the first is far more generally employed. Machines using it, however, usually show very poor regulation at light loads when the suction valve is open during a large portion of the forcing stroke. This is a serious defect in engines employed for the generation of electric power, and is especially troublesome when they are connected to alternators which must be synchronized. The second method, which is as yet of very limited application, promises to overcome this defect completely.

The governors of some large engines, in addition to exercising control of the fuel delivered to the valve, also control the amount and pressure of the blast air and the time of lift of the valve needle, and some very ingenious mechanisms have been devised to discharge all these functions. Except on very powerful engines, however, such elaborate control systems are needless, and introduce complications which are liable to injure the reliability to a considerable degree.

c. Cooling. The cooling of the Diesel engine presents no especial difficulty in the smaller sizes, the ordinary method of water circulation under slight pressure being employed. In fact, it is claimed that the maximum temperatures with the Diesel are less than those with the Otto engine. This arises from the fact that in the Diesel, there is more time allowed for the heat to be transferred to the cooling water because combustion endures for an appreciable period; whereas in the Otto, it is almost instantaneous. The claim seems doubtful, however, especially in the case of high speed engines.

But when cylinders reach a diameter of from 16 to 18 inches in the four-cycle engine and about 9 inches in the two-cycle, it becomes necessary to cool the piston, and this is a matter in which considerable difficulty arises. It has been found that for this purpose oil is a better cooling agent than water, both because a joint may more easily be made oil-tight than water-tight and because an oil leak inside the engine is much less troublesome than a water leak. The introduction of the cooling medium into the piston is accomplished by means of a trombone joint consisting of a long tube attached to the piston and moving with it, which slides in a similar tube rigidly attached to the engine frame. The joint is most carefully packed to prevent leakage, but even with the greatest care and the best material leaks sometimes occur.

f. Lubrication. Owing to the close fitting of parts, which is necessary to contain the high pressures employed in the engine, the lubrication of the Diesel must itself be under considerable pressure, 15 to 20 pounds being a commonly employed figure. The cam shaft usually runs in a bath of oil, and a splash system may be

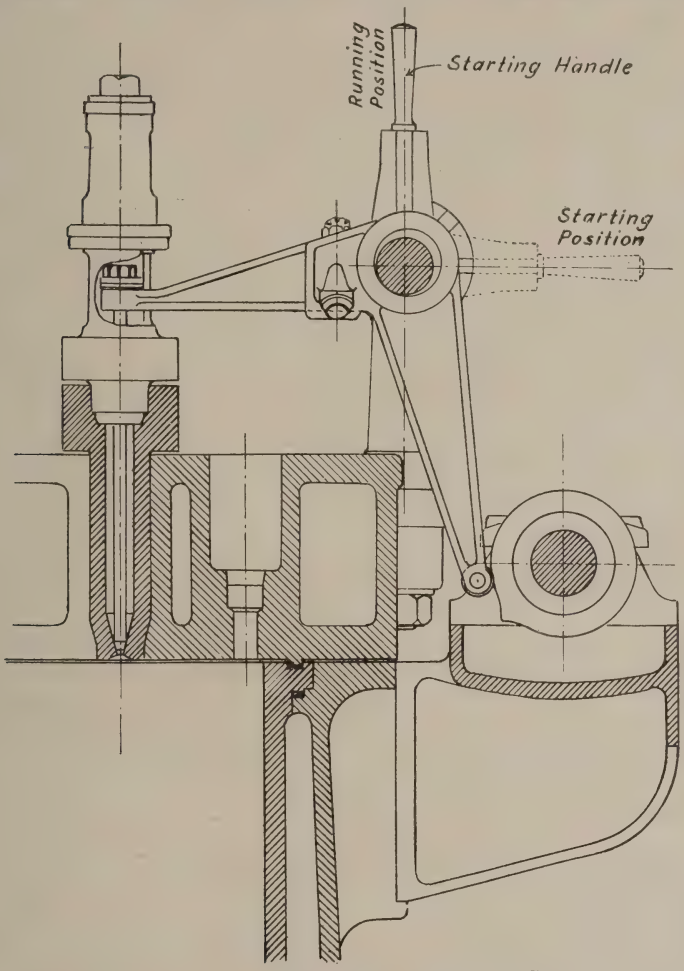


Fig. 10. Fuel Inlet Valve, Lever and Cam.

used to assist in lubricating the pistons, but the pressure system forms the main reliance and should reach practically all parts of the engine.

g. Starting. The engine is started by compressed air admitted through the starting valve. The lever operating this valve is inter-

connected with that operating the fuel valve in such a way that, in a certain position of a controlling handle, the air start valve is operative while the fuel valve remains closed at all times. In another position of the handle the fuel valve operates and the starting valve remains closed at all times. To start, it is therefore necessary to connect the starting valve to a reservoir of high pressure air, bar the engine round until the piston is just past top center, and move the starting handle into the first position. This opens the starting valve and the engine starts under compressed air. After a few revolutions the temperature in the cylinder becomes high enough to fire the fuel charge, the movement of the lever into the second position permits the entrance of fuel, and the engine continues under its own power.

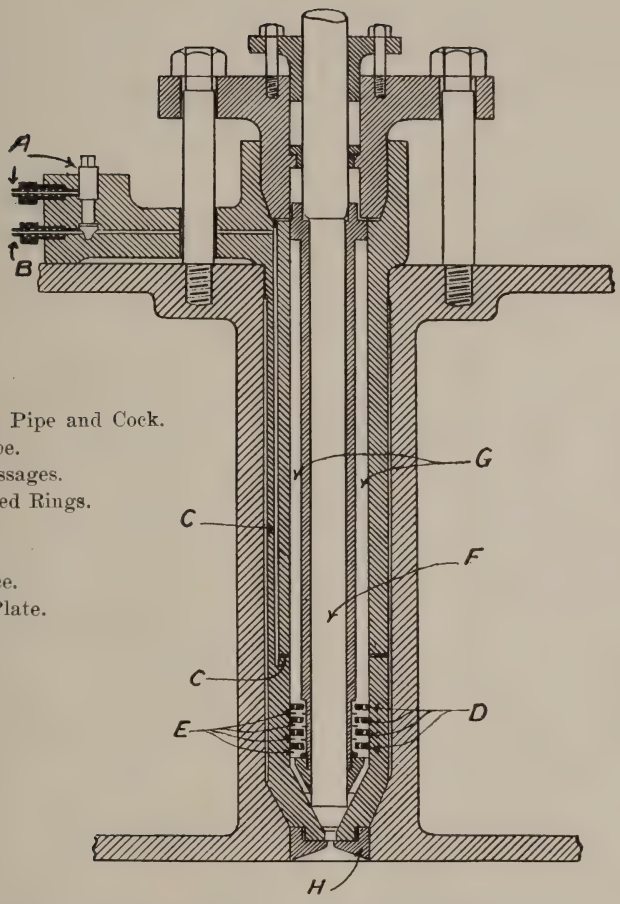
The two-cycle Diesel engine differs from the four-cycle which has just been described both in the valve arrangement and in the provision of a separate air pump for scavenging air. The cylinder has a ring of ports cut in it at the bottom, and a scavenging air valve replaces the air inlet valve. When the piston in its downward stroke uncovers the ring of ports, the burnt gases begin to pass out through them. At the same time the scavenging air valve opens, and air under from 3 to 5 pounds pressure enters the cylinder and forces the burnt gases out before it. By the time that the piston has again covered the ports on its upward stroke, the cylinder is filled with air under scavenger pressure and is ready for compression. This increased pressure at the beginning of compression partially counterbalances the decreased available length of cylinder due to the presence of ports, and enables the two-cycle engine to get into its cylinder at each stroke nearly as much air as does the four-cycle. This has a material effect in increasing engine capacity.

III. FUEL.

The fuels available for use in Diesel engines may, with reference to the sources from which they are derived, be classified as follows:

a. Products obtained by distillation of petroleum.
b. Products obtained by distillation of coal and lignite.
c. Minor oils from miscellaneous sources.

a. Petroleum Products. Under the Diesel method of operation, there is no combustible in the cylinder at any time except when combustion is intended to take place. This renders low flash point no hindrance to the successful use of a fuel, and the Diesel engine



A, Overflow Pipe and Cock.
B, Fuel Pipe.
C, Fuel Passages.
D, Perforated Rings.
E, Spacers.
F, Needle.
G, Air Space.
H, Flame Plate.

Fig. 11. Section of Fuel Valve, Perforated Ring Type.

will run as well or better on the lighter distillates of petroleum, such as gasoline or kerosene, as on the heavier oils ordinarily burned in it. The high price of these light oils, however, discourages their use.

The petroleum product generally employed is the fuel oil commonly burned under steam boilers. This oil distils at from 300° to 360° C. between the kerosenes and the lubricating oils. It has a specific gravity of from .8 to .9, a flash point of 70°-150° C., and a heating value of approximately 18,000 B. T. U. per pound. Being too heavy and inert to be utilized in carbureting engines, and too thin and with insufficient body for service as lubricating oil, this product was confined to use under boilers until the Diesel opened a new field for its employment. It is comparatively cheap, being obtainable in the United States for from three to four cents per gallon, and usually possesses excellent burning qualities. The paraffin base oils of the Pennsylvania fields are admirably adapted for use in Diesel engines. The western and Mexican oils, however, usually contain a large percentage of asphalt which has, until recently, been thought to decrease their availability because of a tendency to gum and clog the fuel valves. Late tests seem to indicate that this tendency is less than has been supposed. Oils containing as much as 21 per cent of asphaltum are said to have been used with success in the California fields, and a very recent article describing tests made in Germany with low grade oils stated that Californian and Mexican crude oils containing between 45 and 50 per cent of asphaltum had been burned for upwards of two hundred hours without trouble. Carefully designed fuel valves, especially those working on the Hesselmann principle of cleaning out the oil space completely at each lift of the needle, will assist the combustion of heavy and dirty oils more than any other factor. The type of valve ordinarily used will require frequent cleaning no matter what the composition of the fuel used may be.

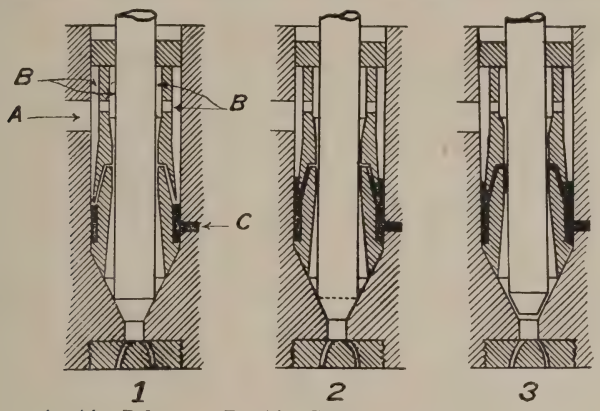
The fuel oil which has just been described is but one of a great number of petroleum distillates that can be successfully employed in Diesel engines. The range of available oils extends from benzine to the semifluid residues which form almost the last product of distillation.

b. Distillates of Coal and Lignite. It may be safely said that practically all the fuel used in Diesel engines in this country at the present time is obtained by the distillation of petroleum. In many portions of Europe, however, where petroleum products are scarce

and costly, other sources of fuel have been developed. Chief among these is the distillation of coal and lignite. Very satisfactory fuel is obtained from both these raw materials.

The lignite oils are reported to be in every way fitted for service in Diesel engines, but their use in Europe has thus far been limited because of their price which approaches that of the petroleum oils. Lignite is found in many parts of the United States, though the development of the deposits has not been extensive. When the demand for petroleum products raises the price to a point where these fields may be profitably worked, greater activity will certainly be displayed.

The utilization of coal by distillation, instead of by direct combustion under boilers, presents so many advantages that it seems bound to become the prevailing practice as soon any great shortage



A, Air Inlet. *B*, Air Space. *C*, Fuel Inlet.

Fig. 12. Section of Fuel Valve, Hesselmann Type.

in the coal supply begins to make itself felt. It is well known that the ordinary processes of combustion under boilers, especially with unskilled firemen, are most wasteful in so far as realizing the full heating value of the fuel is concerned. If, however, the coal be distilled, there is first obtained a combustible gas well suited for service in explosion engines having a thermal efficiency considerably better than that of the steam engine. This gas is also widely used for heating and lighting. Liquids, ranging in weight from the lighter oils similar to benzine to the heaviest tars, are the next products of the distillation. With the exception of the heavier tar oils (which have a market of their own outside the field of power production), all these liquids may be used to excellent ad-

vantage in Diesel engines with a thermal efficiency of between two and three times that of the average steam plant. The solid matter remaining at the end of the distillation is coke, which is largely used in metallurgy and heating. By this manner of handling coal its heat content may be advantageously employed with a minimum of waste.

For a long time, considerable difficulty was met with in burning in Diesel engines the heavier liquid products of the distillation of coal. These oils have high flash points (about 400° F. in some cases) and are very viscous, both of these characteristics tending to hinder complete combustion of the fuel. The difficulties due to high viscosity have been overcome by specially designed valves and by an increase in the blast pressure, while those arising from the high flash point are met by the expedient of arranging the fuel valve and pumps so as to inject a small charge of a light oil of low flash point immediately before the main charge of heavy oil. The combustion of the light oil in the cylinder produces a temperature amply high enough to ignite the heavy oil following it. This procedure is very widely used in Germany, the priming charge, so to speak, being called "*zundöl.*"

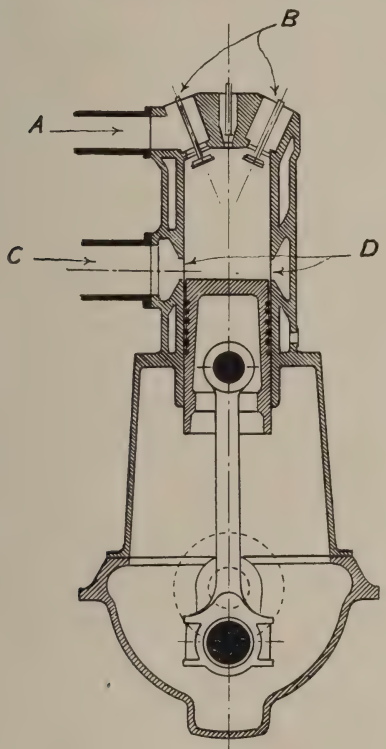
c. Oils from Miscellaneous Sources. Though the sources that have been mentioned furnish by far the greater part of the supply of fuel for Diesel engines, there are many other oils that can be pressed into service. Diesels have been run on animal oils, lard oil and fish oil having been used with entire success. Many kinds of vegetable oils such as castor oil, peanut oil and cottonseed oil may also be employed. Wood distillates have been satisfactorily burned, and alcohol is also suitable for use. These oils are not, of course, used commercially on account of their high price, but in an emergency there would be no objection to their use in so far as successful operation of the engine is concerned.

IV. OPERATION AND PERFORMANCE: COMPARISON OF TYPES.

a. Operation and Performance. Though the Diesel engine is not an involved or complicated piece of mechanism, it is true that its successful operation requires care and skill of a higher order than does that of the ordinary steam engine. On account of its numerous auxiliaries, the Diesel is subject to a considerable variety of troubles. The more common difficulties met with will be briefly enumerated.

As in all internal combustion engines, the supply of cooling

water must be continuous and its circulation free and unimpeded. Ordinarily, little trouble is experienced with the cylinder cooling if proper care be taken to keep the circulating pump in good shape and the water passages open and clear. As has been previously remarked, the piston cooling necessary in engines developing large power per cylinder is a more difficult matter. The problem here is entirely mechanical, consisting in the construction of a sliding joint that will not permit leakage of the cooling medium. These



A. Scavenging Air Inlet.
B. Scavenging Air Inlet Valves.
C. Exhaust Passage.
D. Exhaust Ports.

Fig. 13.—Section of Two-cycle, Single-cylinder Diesel Engine (Piston in Lowest Position).

joints require careful packing and close fitting of the metal parts.

The air compressor sometimes causes trouble through the clogging of the valves by carbon deposited from the air passing through the compressor. Care must be observed to use a minimum of lubricating oil in order that no gumming may take place. The receivers and intercoolers must be efficiently water cooled; as has been stated, this is usually accomplished by making them of small bore

pipe (giving a maximum of radiating surface), and surrounding them with the circulating water.

The pressures in this part of the engine are very high, it being customary to store the air for starting the engine in steel bottles at a pressure of 900-950 pounds per square inch. The average blast pressure when the engine is running light is 750 pounds, and this pressure increases with the load, the usual full-load figure being 825-850 pounds. When using very heavy and viscous oils it is occasionally found necessary to increase the blast considerably, some instances being reported in which a pressure of 1,050-1,100 pounds was required.

The exhaust valve is a rather sensitive spot in many Diesel engines, for the reason that it is subjected to very high temperatures for long periods of time. This valve is especially liable to give trouble in engines of high power per cylinder, since its great size (necessary in order to pass the large volume of the products of combustion) renders it difficult to transfer the heat to those parts of the engine which are in contact with the cooling water. This objectionable feature is completely done away with in the two-cycle engine which, as already stated, uses exhaust ports instead of valves.

There are several points to be considered in regard to the fuel valve, and care must be exercised on handling it since faulty action on its part may have a disastrous result. The packing of the needle must be accurately done so as to prevent leakage from the cylinder, and at the same time to make sure that the needle shall not stick and leave the inlet passage open at the wrong time. Such sticking may lead to preignition, causing very heavy and dangerous explosions in the cylinder. The usual practice is to dissect and clean the fuel valves thoroughly once every two weeks. Engines using valves which completely free themselves of fuel at each injection will require cleaning far less frequently. The kind of fuel used also has considerable bearing on the amount of cleaning to be done, oils heavy with asphalt being especially likely to leave deposits in the valves.

All parts of the engine must be liberally supplied with lubricating oil with the exception of the compressor, where an excess of oil is of more frequent occurrence than a deficiency. The consumption of lubricating oil is usually high.

The test performances of Diesel engines depend to a considerable extent on the type. As with other engines, small machines are

less efficient than large ones, but the point beyond which increase in size does not bring any considerable increase in efficiency is much lower in the Diesel than in steam machines. In general, for the four-cycle engine European builders will guarantee a B. H. P. on 205 grams of oil (.45 pound) having a lower heating value of 18,000 B. T. U. per pound. The two-cycle engine is not quite as efficient as the four-cycle, though the figures for the two machines are much nearer together in the Diesel than in the corresponding types of gas engine. The curve of fuel consumption per B. H. P. hour plotted against load is much flatter in the Diesel than in steam machines, and the efficiency is better maintained at light loads. A thermal efficiency on the B. H. P. of very nearly 30 per cent may be expected with almost all Diesels, and figures considerably better than these have been reached. The lubricating oil consumption is about .0075 pound per B. H. P. hour.

b. Comparison of Types. In inquiring into the relative merit of the various types of Diesel engine the system of operation will first be taken up. The four-cycle engine in almost every case gives better thermal efficiency than the two-cycle. The difference amounts to 3 or 4 per cent, and is accounted for by the fact that, due to the entrance of scavenger air, expansion in the two-cycle engine must be more or less incomplete, and by the additional consideration that power is required to drive the scavenger pump. The two-cycle eliminates a very fruitful cause of trouble by using exhaust ports instead of exhaust valves which are, in the four-cycle type, very liable to become foul and overheated. For a given power two-cycle engines are lighter and occupy less space—their cost should also be correspondingly less. Large power can be obtained in the two-cycle without the necessity of using a multiplicity of cylinders. Finally, the two-cycle engine can be more easily reversed—a very important point in marine work.

Notwithstanding the above considerations, most of which favor the adoption of the two-cycle type, the four-cycle is still the more extensively built. This is because its design is more simple, its construction is better understood by most manufacturers, and more experience has been had in its use. Whether the four-cycle will be able to maintain its present position is, however, very doubtful.

The question of speed is one in which the designer's judgment must be trusted to achieve the most desirable compromise. High speed offers the important advantages of decreased size, weight.

and cost for the same power. Speeds have been increasing in recent designs, and have been set as high as 500 R. P. M. for engines developing 200 B. H. P. This figure, however, seems to be very near the limit for reciprocating machinery, unless in small sizes. Many think that 400 R. P. M. would be safer, and European practice shows a tendency to keep the speed below this point. Most of the experience with Diesel engines has been gained with machines running at 200 R. P. M. or less, but engines of higher speed are now coming into favor.

The construction of the double-acting Diesel presents many difficult problems. Chief among these is the task of making the piston rod stuffing boxes tight against the high pressures employed. The cooling of piston rods and pistons, which is never easy even in single-acting engines becomes much more difficult when almost double the amount of heat has to be transferred from the parts in question. Notwithstanding these facts, the increasing demand for power in large quantities has required the construction of double-acting engines to meet it, and the objections to this type are being rapidly overcome. A late development takes the form of a single cylinder, two-cycle, double-acting engine which on test showed considerably more than 2,000 B. H. P. It seems assured that the large engines of the future must be double-acting.

For a long time the Diesel was built of the vertical type exclusively, and this is still by far the more common kind. The horizontal engine requires more floor space, but also demands less headroom—which in some instances may cause it to be especially suited to the requirements of the location. Due to the pressure of the piston against the lower side of the cylinder, unequal wear of the cylinder walls may take place in the horizontal engine. It must also have its fuel valve in a horizontal position, whereas many varieties of valve can not be satisfactorily used when so placed. Its thermal efficiency is slightly less than that of the vertical engine. On the other hand, there is less vibration in the horizontal type, and the mechanism can be more readily dismantled and piston and rod removed from the cylinder. The engine is lighter than the vertical, and is less expensive to build. The last is a powerful argument in the United States, where the high first cost has had much weight in delaying the introduction of the Diesel, and it seems quite possible that the horizontal will prove to be the popular type in this country.

V. APPLICATIONS: COMPARISON WITH OTHER FORMS OF PRIME MOVER.

It is proposed to investigate the application and merits of the Diesel engine in three fields, namely:

a. Electric Power Generation.
b. Marine Propulsion.
c. Railroad Traction.

The three divisions of the subject will be considered in the order named.

a. Electric Power Generation. Present methods of utilizing power show a marked tendency toward the interposition of an electric link between the prime mover and the machine actually doing work. The only fields in which this tendency is not notably apparent are those of railroad traction and marine work, and indications are not lacking that future development in both these lines will involve the use of electric power. This being the condition of affairs, the best criterion of the value of a prime mover must be its adaptability for use in driving generators.

In considering the Diesel for such use, a comparison with the other forms of prime mover at present employed in this work must be made. Perhaps the latest authoritative expression on this subject was contained in a paper on "The Present Status of Prime Movers," presented before the American Institute of Electrical Engineers in June, 1914, by its committee on prime movers. The substance of the committee's report was as follows:

1. Except for small, isolated stations, the reciprocating steam engine has become practically obsolete for use in central power stations.

2. The steam turbine is at the head of the list of available prime movers at the present time. It is capable of being built in the largest sizes required for any purposes. Its first cost per kilowatt is considerably less than for any other type of prime mover, being given as $7.50 per kilowatt for a 20,000 kilowatt installation. The thermal efficiency in the newest and largest turbines is approximately 24 per cent, which is better than the best figure attainable by gas engines, and is only exceeded by that reached by the Diesel engine. The steam turbine has high overload capacity, usually 50 per cent in excess of the load at which the best steam economy is obtained. In fact, it is stated that the generating unit should be so proportioned that the best water rate of the turbine occurs at

about 80 per cent of the continuous capacity of the generator. The reliability is greater than that of any other form of prime mover.

3. The gas engine is now built in America in sizes up to 6,000 B. H. P. Natural and blast furnace gases are used for fuel in 75 per cent of the gas power installations in the United States, the producer being the source of fuel in the remainder. The efficiency of the gas engine increases with the load, and the rating is therefore made very close to the limiting capacity, allowance being usually made for an overload of 10 to 15 per cent. The thermal efficiency on the kilowatt-hour basis is nearly 24 per cent in the largest sizes. The engine is very heavy, averaging 700 pounds per kilowatt for engines of 750-kilowatt capacity. A curious feature is that this average weight increases with increased engine power. The average cost per kilowatt is $72 for engines of 1,000-kilowatt capacity.

4. The Diesel engine has made very slow progress in America. The thermal efficiency on the kilowatt-hour basis is about 28 per cent, the oil consumption being about .65 pound. The engine is heavy, averaging 700 pounds per kilowatt in sizes of 140 kilowatt; and, although the weight shows a rapid increase with the size up to this point, the rate of increase is much smaller beyond it for the engines considered. The largest Diesel operating in the United States is of 450 B. H. P., while in Europe much larger sizes have been developed.

In summarizing the results of their investigation into the status of the available heat power prime movers, the committee presented the following figures and conclusions:

In arriving at the cost of power two sets of units were considered. In the first were included a 750-kilowatt steam turbine costing $15 per kilowatt, a gas engine unit of 750 kilowatts costing $73 per kilowatt, and two Diesel units (each of 375 kilowatts) costing $95 per kilowatt. Investment costs (to include interest, taxes, insurance, and depreciation or amortization) were taken at 11 per cent for the turbine, and 15 per cent for the gas engine and the Diesel. Coal was assumed to cost $3.00 per ton, and oil 4 cents per gallon. With these figures as starting points the costs per kilowatt-hour output at full load were, for the turbine, investment $0.0002, fuel $0.003, total $0.0032; for the gas engine, investment $0.0013, fuel $0.002, total $0.0033; for the Diesel, investment $0.0017, fuel $0.0033, total $0.005. A similar comparison was made for a plant of 20,000-kilowatt capacity, the units considered being one steam turbine of 20,000 kilowatts costing $7.50 per kilowatt, ten 2,000-kilowatt gas engines costing $65 per kilowatt, and

forty 500-kilowatt Diesel engines costing $85 per kilowatt. Using the same prices for fuel as before, we now find the cost per kilowatt-hour to be for the turbine, investment $0.0001, fuel $0.002, total $0.0021; for the gas engine, investment $0.0012, fuel $0.002, total $0.0032; for the Diesel, investment $0.0015, fuel, $0.0033, total $0.0048.

The committee remarks that these figures show the gas and oil engines to be practically out of the running. The turbine gives greater reliability of service, and smaller maintenance and repair costs. The turbine also has a great advantage in space economy, the statement being made that the number of kilowatts per square foot of floor space is about 12 for the turbine and about .6 for the oil engine and gas engine.

The report just summarized makes out a rather unfavorable case for the Diesel, but there would seem to be certain considerations not taken into account by the committee which should offer a little hope to the believers in the oil engine. These considerations may be stated as follows:

1. The prime movers considered by the committee were all of American manufacture. For the last ten years the focus of the attention of American designers has been the steam turbine, and it has, in consequence, been brought to a very high pitch of perfection —so high, in fact, that any further improvement of consequence seems doubtful. Much labor has likewise been devoted to the gas engine, though its development has not been carried out as thoroughly as has that of the turbine. The Diesel, however, has attracted but little interest in this country until very recently; and the power stations already installed, which were, of course, the only ones that the committee could consider, are for the most part productions very inferior to current European practice. The engines are of the four-cycle type, very heavy and cumbersome, and of slow speed. Moreover, they are of small size compared with those at present built and building in Europe. In short, the committee's comparison is between a steam turbine that is as good as has ever been built and probably very nearly as good as ever will be built, and a Diesel much inferior to engines actually in service. No allowance is made for future development in the Diesel, which is certain to be both rapid and extensive.

2. In connection with the cost data submitted, the committee makes this statement: "As it is not within the scope of this paper to include operating and maintenance costs *for the entire plant*, investment costs *of the units* and fuel costs are shown plotted in

conjunction with the percentage of normal full-load rating for making an economic study of the various types of prime movers." Observing this limitation, the first cost figures submitted are for prime mover and generator only, omitting all auxiliaries. The investment costs and required floor space are also given for prime mover and generator alone. By presenting figures on this basis, one of the most important advantages of the Diesel over its competitors is nullified.

The heat used by the turbine must be furnished to it in the form of steam, the generation of which requires an extensive and costly provision of boilers, boiler feed apparatus, furnaces, stokers and other auxiliaries. Furthermore, to get the best efficiency from the turbine, a condenser and associated apparatus must be installed. To obtain a fair comparison between the different power producers, the turbine should be charged with both the extra cost and the extra floor space made necessary by these essential parts of the plant.

The same reasoning holds good for the gas engine which, unless run on natural or blast-furnace gas, requires a producer to prepare the fuel for use. The majority of power plants are not situated where these gases are available.

The Diesel, however, is a self-contained unit to a much greater degree than either of its competitors. Its fuel is burned directly in the cylinder without previous preparation of any kind. Its auxiliaries are all driven by the engine itself, and the power they require must necessarily be considered in the figures given for the cost of the output per kilowatt-hour. All it demands, in addition to fuel, is cooling water and lubricating oil, air being the only other requisite.

To show the bearing on power costs of the limitation observed by the committee, there is inserted a table extracted from the report in question. This table refers to the 20,000-kilowatt station considered, and is as follows:

Prime mover.	Total investment cost per kilowatt.	Cost of prime mover and generator per kilowatt.	Percentage of total investment.
Steam turbine	$65.00	$7.50	15
Gas engine (producer gas)	140.00	65.00	46.5
Diesel engine	120.00	85.00	71

It now seems that after we have bought our steam turbine and generator we are the possessors of 15 per cent of a power plant, and have to spend an additional sum of $57.50 per kilowatt capacity before we can deliver power to our switchboard. But when we have bought our Diesel engine and generator we have 71 per cent of a power plant, and can deliver power with the expenditure of $35.00 more per kilowatt. The ratio of cost between the Diesel and the turbine appears to be 2 to 1 instead of 11 to 1 as was indicated by the original figures. Moreover, the Diesel plant considered is of a most expensive type, being made up of a multiplicity of small units which would be costly in themselves and require a great amount of space for their accommodation.

3. It would appear that the function of a power plant is to transform the heat energy in the fuel into electrical energy at the switchboard, and that the figure given for the efficiency of this transformation should include all intermediate processes—not merely a part of them. The 28 per cent given by the committee for the Diesel is a true measure of this efficiency of transformation, since the heat content of the fuel is measured as it is fed to the engine, and the committee's figure is obtained by taking the ratio of the output of heat units delivered to the switchboard in the form of electrical energy to this heat input.

The 24 per cent efficiency claimed for the latest turbines (which is a very high figure and is only obtainable with machines of the very best type) is calculated on the heat units actually delivered at the throttle. By so doing all consideration of boiler, auxiliary and transmission losses is omitted. To put the turbine on the same basis as the Diesel its 24 per cent must be reduced by at least the losses occurring in the boiler, and this still leaves out of account the steam consumed by the various auxiliaries which are necessary. Taking the boiler and grate efficiency as 70 per cent, which is a higher figure than is obtained in practice except on specially conducted tests, the overall efficiency of the turbine plant becomes $.7 \times 24 = 16.8$ per cent—not quite as favorable a showing.

4. It is true that the attendance required for the Diesel engine is of a higher quality than that needed for the turbine. On the other hand, the number of attendants necessary is very considerably less, and the labor cost per year will probably be lower for the Diesel plant than for the turbine plant of equal capacity. Unfortunately, no figures from actual experience bearing on this point are at hand except some from advocates of Diesel engines, which it is not

desired to present because of their origin. The installation of mechanical stokers will improve the showing of the steam plant in this respect, but will also entail a considerable increase in the first cost of the plant.

5. The steam machinery has to carry an inevitable amount of stand-by losses which are entirely absent with the Diesel. The gas producer is also subject to this important source of loss. Such losses have been estimated to be as high as 30 per cent, though this figure is probably excessive.

6. A most important point which is usually lost sight of in comparing the Diesel with other prime movers is the following:

The relative merits of installations are generally determined by comparison of test data. Tests are ordinarily run with all apparatus in its very best shape to do efficient work, with careful supervision of all operations, and with every precaution taken to insure making the most favorable showing possible. Measurements of fuel consumption in steam plants continuing over considerable lengths of time, and setting forth actual operating conditions rather than test conditions, invariably show a large falling off from the efficiency recorded on test. The same is true of the gas producer. The Diesel installation, however, shows very little decrease in efficiency even after long periods of time.

A power plant owner is most interested, not in what his plant will do on test, when new and handled with the most expert and painstaking care, but in what his costs are going to average per year under actual running conditions.

A Diesel plant may be expected to give results in daily operation very close to its test figures, while both steam and gas producer plants fail when in service to approach their performances on test.

7. In Germany, and in general throughout Europe, the Diesel is considered to be fully the equal of the steam turbine in regard to reliability. Whether this is true in America at the present time is doubtful—it certainly was not the case a few years ago. There is reason to believe, however, that with the increase in America of attention to Diesel construction, engines will soon be improved to such a point that they will compare favorably with those of foreign manufacture. In Europe no extra insurance premium for fire risk is required for Diesel plants, and they are thought to be as safe as any other form of prime mover.

The steam turbine is at present the pre-eminent form of prime mover for central station work, and is likely to remain so for years

to come. Enough has, perhaps, been said to show that the development of the Diesel engine has disclosed possibilities which may, in future, make it a very serious rival of the turbine for even the largest powers.

b. Marine Propulsion. The field of marine propulsion is one in which many attempts have been made to introduce the Diesel engine; and the results obtained give promise that a large measure of success will in future attend these efforts. In comparison with the usual reciprocating engine installation, many points about a Diesel marine plant are very attractive—among them the following:

1. No fire room and stokers are required, resulting in saving of both personnel and space.

2. No boilers are used and no smoke is developed.

3. No fresh water is required—salt water can readily be used for cooling.

4. The engine room space is from 30 to 50 per cent smaller and is more easily kept clean.

5. The advantages of oil fuel over coal for marine purposes are so marked as to have led to the adoption of oil for use under boilers in most of the navies of the world. Among these advantages are the following:

a. Fuel can be taken aboard much more rapidly and with less labor, and can readily be transferred between ships at sea.

b. Steam for full power is maintained as readily as for low power. There is no reduction in speed due to dirty fires, difficulty of bringing coal from remote bunkers or exhaustion of the fire-room force.

c. Coal and ash handling gear is eliminated.

d. The supply of fuel to the furnaces is susceptible of much more delicate and prompt control.

e. Remote parts of the ship and constricted spaces useless for coal storage may readily be employed to carry fuel oil.

f. A ton of coal occupies 40 cubic feet, while a ton of oil occupies 35 cubic feet. Weight for weight, therefore, the storage value of oil compared with coal is as 11.5 to 10. Since the relative heating value of oil compared with that of coal, weight for weight, is as 14 to 10, it appears that for steam production aboard ship the value of oil to coal is as 16 to 10 if storage capacity be taken into account.

6. When, in addition to the considerations mentioned in 5*f* above, it is remembered that the efficiency of the Diesel marine engine compared with that of a good boiler and triple-expansion engine installation is as 25 to 10, it seems that, considering fuel storage, calorific value of fuel, and engine efficiency, the Diesel plant will furnish four times as much power as the coal steam plant.

On the other side of the argument, it must be said that oil is much more expensive than coal, weight for weight; that the first cost of the Diesel installation is considerably greater than that of the steam plant; and that the steam engine has proved its reliability by many years of service, while the Diesel is a comparative newcomer into the field.

At the close of 1912 there were some five hundred Diesel vessels in use. No figures as to the number and size of the vessels at present in service are available, but it is known that engines developing as much as 6,000 horsepower have been constructed for marine work, and engines of 2,800 horsepower have been installed and are in successful operation.

The tendency in marine work, as in stationary work involving high power, is all in favor of the adoption of the two-cycle principle, and single-acting engines of this type as large as 6,000 horsepower are considered quite practicable. For even higher powers the two-cycle double-acting engine will be used, and one machine developing 12,000 horsepower has already been built. The two-cycle is especially adapted to marine work, not only because of its advantages in decreased size and weight but also because of the ease of reversing, which is of course a necessity in marine practice. For submarines the Diesel is almost the only engine to be considered, and its adoption for this use seems likely to become well-nigh universal.

c. Railroad Traction. In connection with land transportation, the development of the Diesel has thus far been comparatively slow. A Diesel-electric car has for some time past been in successful use on the State railroads of Sweden, and Diesel locomotives have been reported to be under construction in Switzerland, though no accounts of their performances are at hand. The use of the electric link in designs of this kind seems essential, the product being somewhat similar to the gasoline-electric cars which are used to a considerable extent in various parts of this country. No attempts to develop the Diesel for these purposes have yet been made in the United States, and, as the reports from Europe are

of a fragmentary nature, very little can be said concerning the progress in this field.

VI. PROBABLE FUTURE DEVELOPMENT.

It is not thought that, for large central station work, the Diesel engine will seriously threaten the present supremacy of the steam turbine for many years to come. This is especially true in respect to the United States, where coal is cheap and where the high first cost of Diesel equipment seems to have a very discouraging effect on the prospective buyer. The turbine may, however, be hard pressed in Europe where conditions are very different. But as an auxiliary to the main units of a central station, for the purpose of caring for sudden and unexpected peak loads, the Diesel should be of great value. Such loads are often imposed on city plants by the occurrence of storms during the hours of daylight —the accompanying darkness making necessary the use of lights throughout the city. These peaks may occur with not more than ten or fifteen minutes warning, and there is an obvious advantage in having at hand one or two Diesel units which can be made to take full load three minutes after being started from a stone-cold condition. The Diesels might also be so proportioned as to carry the small load occurring from midnight until six o'clock, which would considerably underload one of the main units.

It has been suggested, as a possible result of advance in Diesel development, that the present large central stations may, in future, be replaced by smaller scattered stations containing Diesel units. This would result in important savings in transmission. The idea giving rise to this suggestion is the fact that the Diesel engine seems to reach its maximum efficiency in sizes of 100 kilowatts and little, if any, efficiency is gained by increasing the size of the units beyond that point. This is far from being the case with the turbine, which does not approach its maximum efficiency until units of from 1,500 kilowatts to 2,000 kilowatts are reached.

A very good illustration of the work for which Diesels are peculiarly adapted, and for which their use will undoubtedly increase, is found in the electrical installation of the battleship *Moreno*, recently completed in this country for the Argentine Republic. This vessel (of about 28,000 tons displacement) is provided with four horizontal 375-kilowatt turbo-generators for power purposes, but for service in ports, when most of the fires are drawn and little or no steam is being generated, two 75-kilowatt Diesel sets are provided.

For marine purposes it seems probable that the use of the Diesel will be greatly extended. Its merits in comparison with the usual reciprocating engine plant have already been given. The steam turbine has been used to a large extent in vessels of great power and speed, and is now the standard equipment for such work. But on account of the high rotative speed of the turbine, it is unsuited for cruising and for vessels which habitually move at a comparatively slow rate. Various styles of reduction gearing have been proposed for use with the turbine, but it would appear that none have been thoroughly satisfactory. It is now proposed to insert an electric link between the turbine and the propeller in order to utilize the high speed of the turbine to best advantage, and at the same time to have ready control of the speed of the propeller shafts. This electric link has already been very successfully tried out in the United States naval vessel *Jupiter;* and when bids were recently opened for three new battleships for the Navy, the lowest bid received was on a vessel with electric-propelling machinery. This proposal was submitted by the New York Navy Yard, and, as the ships in question are of 32,000 tons displacement, it may readily be inferred that the engineers of the Navy consider the *Jupiter* experiment ample basis on which to proceed to the widest use of the electric link.

While the adoption of the electric drive makes it possible to use the turbine for all classes of marine work, it should be noted that the Diesel may be used with equal facility without the interposition of the electric link. Though its advantage in fuel economy will probably be somewhat reduced, that in space economy becomes greater than before. The turbine must also bear the cost of the electrical installation. This last factor should go far toward putting the two systems on an equality in regard to first cost.

The employment for which the Diesel is best suited will probably be found in vessels of medium size and power. In such ships cargo capacity is of prime importance, and the space saved by the elimination of boilers and condensing plant, to say nothing of the reduced size of the engine room itself, will be of great value. This advantage should go far towards compensating for the higher first cost of the Diesel machinery.

Diesel locomotives and motor cars are being developed at the present time, and further progress may soon be expected. As has already been stated, it seems probable that most of these machines will be provided with the electric link between the engine and the axle in order to secure both flexibility and power to start under

heavy load. Some work has been done in Germany looking toward the employment of the Diesel in automobiles; but on account of the very considerable weight per horsepower, the amount of skill and care required for successful operation, and the high pressures involved, this will probably be one of the last uses to which the engine will be put.

Throughout this paper it has been stated that the high first cost of the Diesel is one of the principal obstacles to its success. It seems proper here to point out that this difficulty is one that may to a large extent be removed in the near future. The development of the engine has proceeded with great speed and is still under way. It has been impossible to standardize the construction, for the reason that improvements have been made so rapidly as to render a machine obsolete within a comparatively short time after leaving the maker's hands. As a consequence, almost every engine has practically been built to special order, and the cost has naturally been excessive. As the industry progresses toward a settled and definite basis, standardized lines of engines will make their appearance and considerable reductions in cost should accompany that event. This must not be taken to mean that the first cost of the Diesel will ever be brought as low as that of the steam engine. The nature of the engine demands the best of material and workmanship to attain success, and it is a settled fact that the Diesel can not be built both successfully and cheaply. But the present prices for the engines are too great, and should suffer material reduction.

In the United States, the Diesel will probably make very much slower progress than in Europe. In the principal manufacturing portions of this country coal is abundant and cheap, while the present boom in water-power development provides a second source of low-priced power. Against such competition it will be hard for the Diesel to make its way. On the Pacific Coast its chances seem considerably better. Here, too, the competition of large water-power developments must be met, but coal is practically out of the running, being supplanted almost entirely by fuel oil. In a locality where oil is used very extensively as fuel under boilers and for general heating purposes, the Diesel must seem very attractive as a means of power production by which greatly increased efficiency may be obtained, and recent information from California indicates that the oil industry there is taking active steps toward thoroughly trying out the engine's possibilities. It is to the Pacific Coast that the Diesel must look for the opportunity to show the power consumers of this country the benefits that may be gained by its use.

Butterfly Valve Experiments.

BY

Lieut. Col. CHARLES KELLER,
Corps of Engineers.

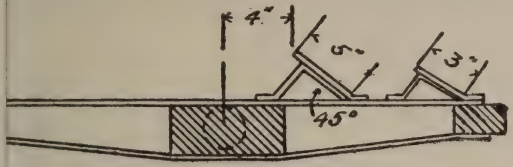
The Warrior coal fields of northern Alabama contain a large supply of steam-coal. As compared with the best West Virginia coals, that of the Warrior fields is relatively low in fixed carbon, and it is much softer. Nevertheless, much of it is of very high quality, running well above 14,500 B. T. U. in heat value, with an ash content sometimes lower than 4 per cent.

Some of the very best of this coal is close to the banks of the Mulberry and Locust Forks of the Black Warrior River, and while its relative softness renders pecularily desirable water transportation with its immunity from the continuous jarring experienced on the railroad, the possibility of considerable saving in cost of transportation by the use of the water route, caused the desirability of furnishing dependable facilities for navigation to be discussed early in the history of the State, and in more recent years a channel connecting the coal district with tidewater at Mobile has been provided by the United States.

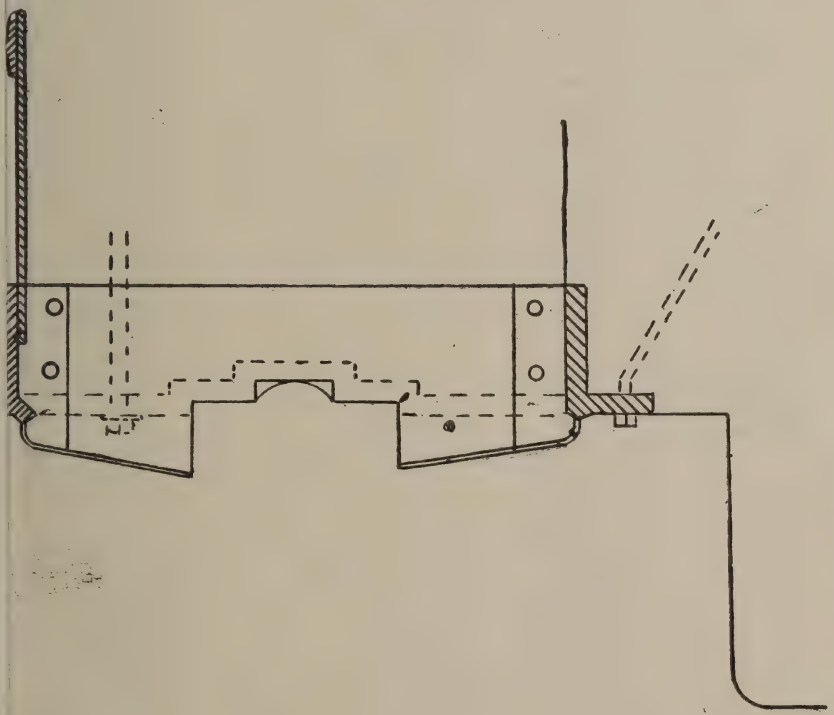
A general description of this improvement is to be found in the paper on "Improvement of the Black Warrior, Warrior and Tombigbee Rivers in Alabama," by U. S. Assistant Engineer R. C. McCalla, in Vol. 28, No. 4, Transactions A. S. C. E., and in the recent paper by U. S. Assistant Engineer Mower on "Locks and Dam No. 17, Black Warrior River" in PROFESSIONAL MEMOIRS, No. 33, so that no further description seems called for.

The locks have normal lifts as follows: Locks 1 to 12 inclusive, 10 feet each; Lock 13, 12 feet; Locks 14 and 15, 14 feet each; Lock 16, 21 feet, and Locks 17A and 17B (a pair in flight), 31.5 feet each. All the locks are filled and emptied through culverts, which opening from the gate recesses, by-pass the gates. The filling culverts communicate with the lock chamber through ports at the floor level, the emptying culverts discharge below the lower gates through single openings.

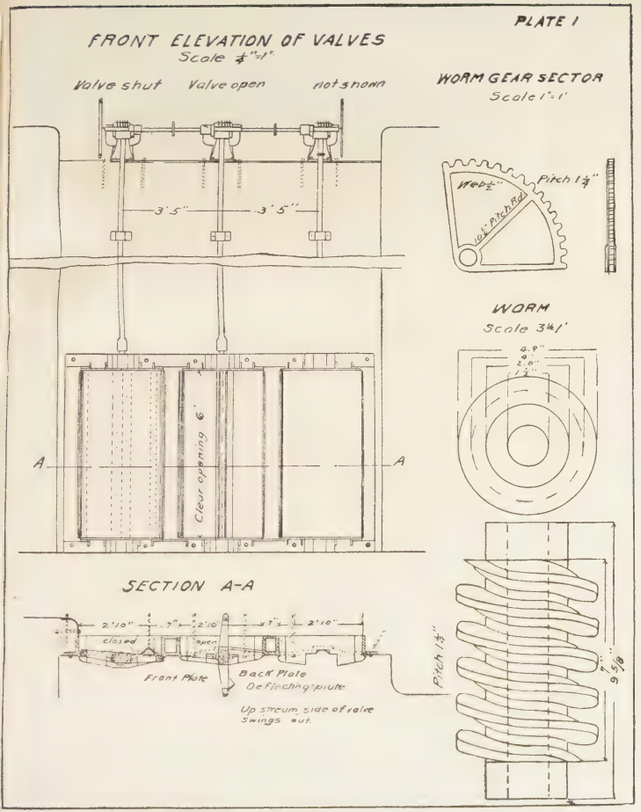
PLATE II

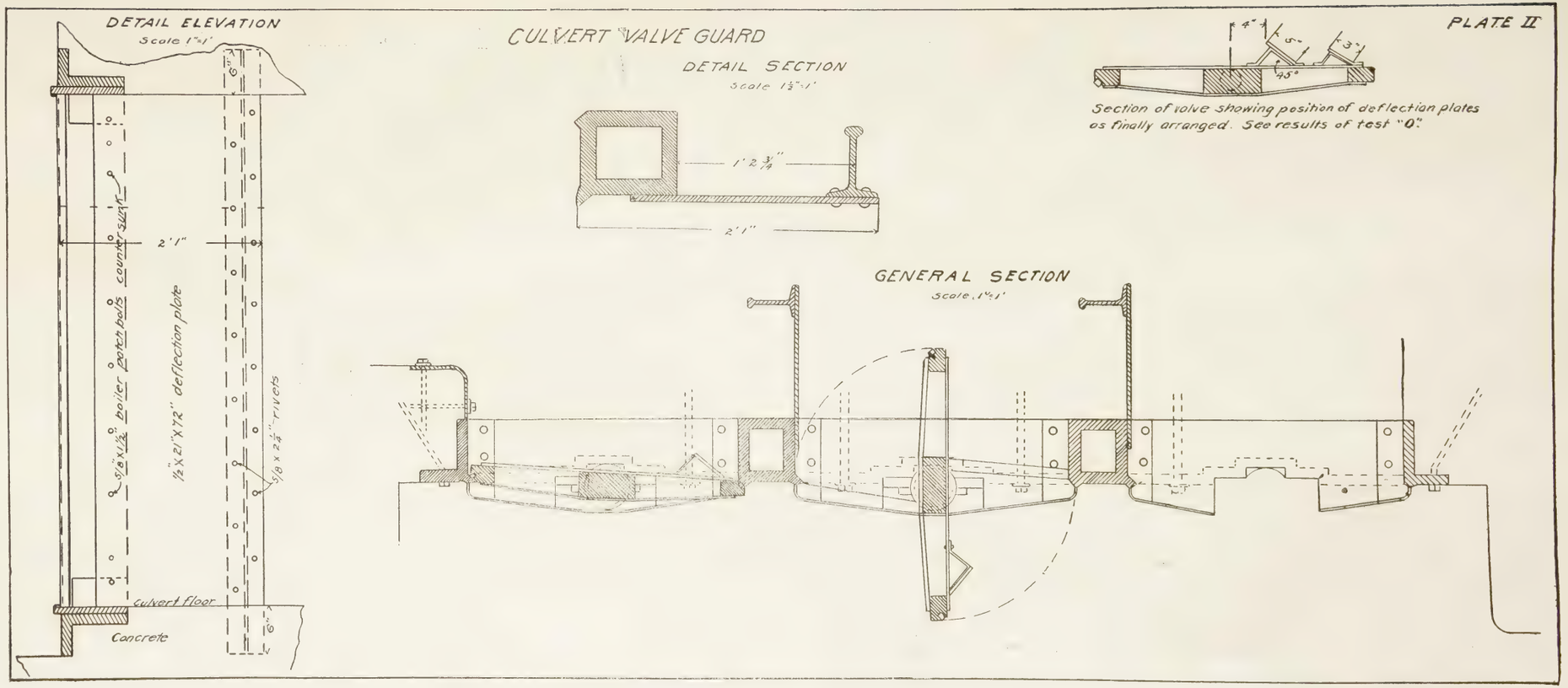


tion of valve showing position of deflection plates
inally arranged. See results of test "O."

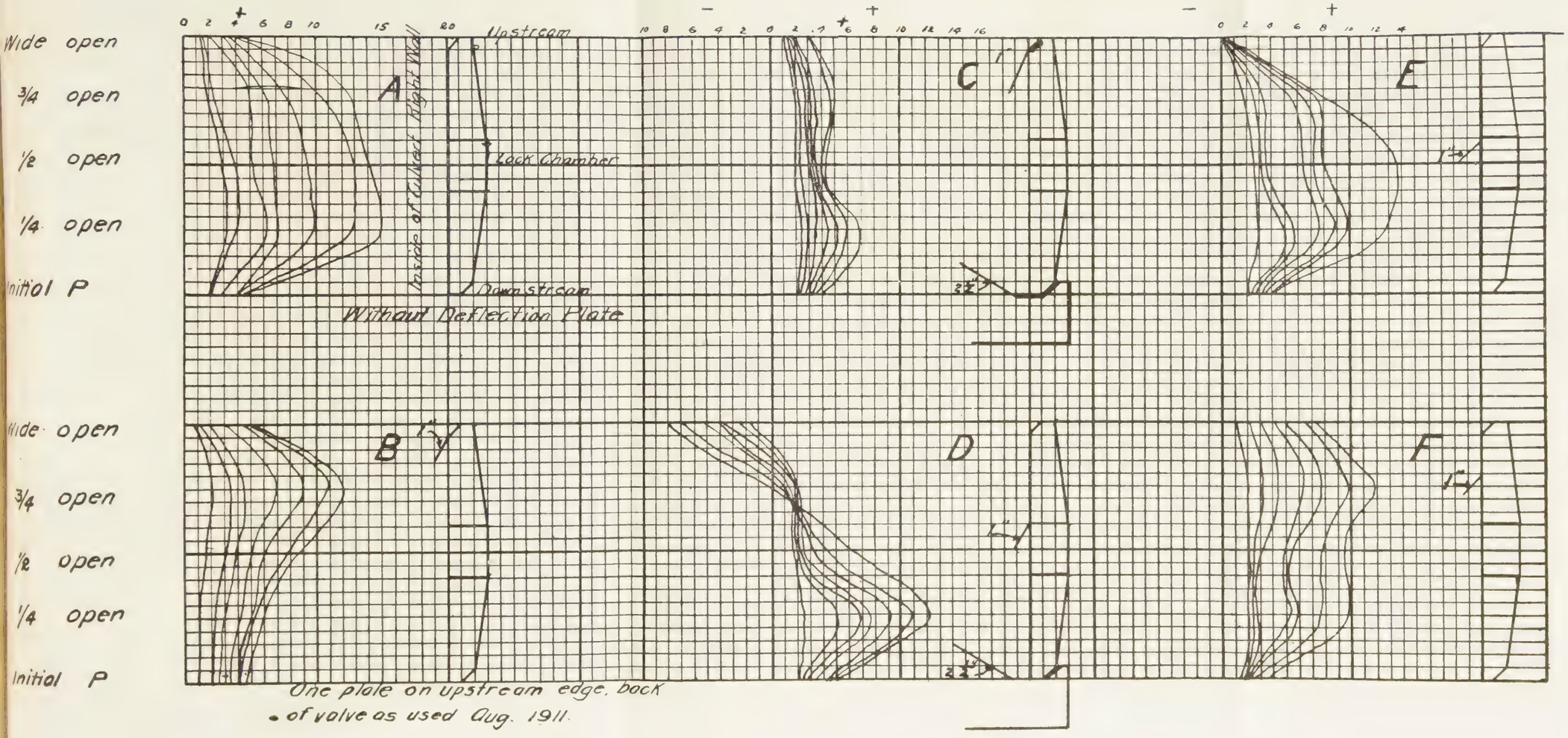


PLATES I and II.





PLATES I and II.



Note: Each set of curves shows variation of pressure on 18″ operating lever in opening valve against heads of 1, 2, 3, 4, 5, 6, and 7 feet.

Model valve built one-fourth the dimensions of actual valve in use and deflection plates 1″ and $1\frac{1}{4}$″ represent 4″ and 5″ plates on full-size valves.

EXPERIMENTS WITH MODEL VALVE.

To determine size and position of deflection plates corresponding to minimum force for operating.

PLATE III a.

Except Locks 17A and 17B, which have cylindrical valves, all culverts are controlled by butterfly valves, either in pairs or in sets of three. These valves are set with their axes vertical, and are operated by hand through the simple worm and sector shown on Plate I. Where the valves are in sets of two, their worm shafts are united by a flange coupling, and the valves are simultaneously operated by a single attendant. Where the set consists of three valves, one of them was usually operated separately, thus requiring either two attendants, or, preferably, two operations on the part of the same man.

While certain difficulties in their operation led to the tests and changes hereafter described, it may be stated that butterfly valves have given entire satisfaction, and that, from our knowledge of them, there is no reason to doubt that they are suitable, and because of their low first cost and cost of repair probably preferable, for lifts up to 25 feet.

The first eight locks built on the Warrior and Tombigbee rivers were equipped with pairs of butterfly valves, 6′ 0″ high, 2′ 10½″ wide, operated from the tops of walls. At three of these locks each valve was set in an independent frame, and operated as a single unit by a simple lever keyed to the top of the shaft. At the other five locks the valves were operated in pairs by the worm and quadrant gear previously mentioned. With an applied force of 100 pounds, this gear exerted a moment of 41,500 inch-pounds about the two shafts, or 20,750 inch-pounds about each shaft. This type of valve and gear was satisfactory for lifts of 10 feet or less, but was rather slow, the filling time being eight minutes with chamber area of 17,200 square feet. Later locks have been designed with three valves to each culvert set in a single frame and separated only by 7-inch cast-iron columns, all as shown upon Plate I.

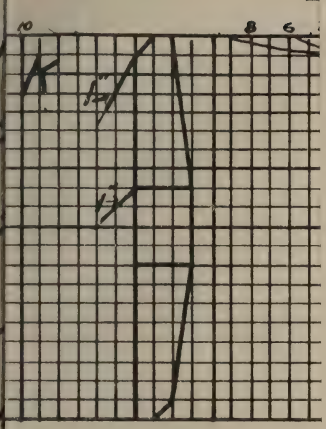
With three valves thus connected to a worm and gear device, it was found that the total moment which one man could exert was 58,020 inch-pounds, or 19,340 inch-pounds to each vertical shaft, not enough, with heads of over 10 feet, to permit a single operator to open the set of three valves to normal position. Only by disconnecting the gear of one of the valves and operating it singly could the set be opened by one man, and any appreciable saving in time of filling be derived from the third valve.

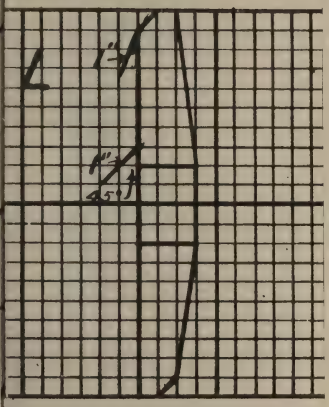
The difficulty of operating three valves keyed to a single handwheel was attributed to two causes. When two or more valves are operated simultaneously, water deflected from the first or upstream

valve, when partly open, comes into contact with and tends to close the second valve. The stream deflected from the latter offers a similar resistance to the opening of the valve next downstream. Secondly, the valve divides the stream moving into the valve opening, and the portion of the current downstream of the valve is deflected by the valve body. The eddy-motion thus caused may produce a reverse pressure on the in-moving or downstream side of the back plate of the valve near the culvert floor, thus also tending to close the valve.

The retarding influence of one valve upon another has been overcome at some of the locks of 10 and 14-foot lift by installing steel partitions between the valve openings, as shown on Plate II. At Lock 16 (21-foot lift) reinforced concrete partitions were provided during the construction of the lock.

In August, 1911, under the auspices of Assistant Engineers G. K. Little and H. C. Mower, assisted by other engineering employees of the Mobile District, a series of tests was made to ascertain the effect of various arrangements of deflection plates applied to butterfly valves of the kind previously described. A quarter-size valve (18″ high and 8⅝″ wide) was carefully built of steel. This valve was provided with a frame of oak and was operated with an 18″ lever. The boxes for the ¾″ shaft were bushed, and a sensitive scale was used in determining the force necessary to be applied to the lever to overcome the resistance to opening at initial, ¼, ½, ¾ and normal positions. The pressure necessary to close the valve when certain arrangements of deflecting plates had caused the valve to spring open, was also measured, the results being shown in figures "D" and "J" of Plates IIIa and IIIb. The valve was installed in a forebay in the form of a tank 4′ 8″ x 14′ in horizontal dimensions, and 10′ deep. This tank had a single rectangular opening at the bottom which was closed by the model valve already described. An 8″ centrifugal pump was used to fill the tank, and an adjustable weir below the valve permitted any head from zero to 7 feet to be maintained. The force applied to the lever at the top of the valve shaft was measured by means of a spring balance, and readings of the spring balance were taken for five positions of the valve operated under heads varying by even feet between 1 and 7 feet, for each arrangement of deflection plates. The results shown on Plates IIIa, IIIb and IIIc are the readings of the spring balance corrected for frictional resistance, and are believed to be self-explanatory.


10
8
6
K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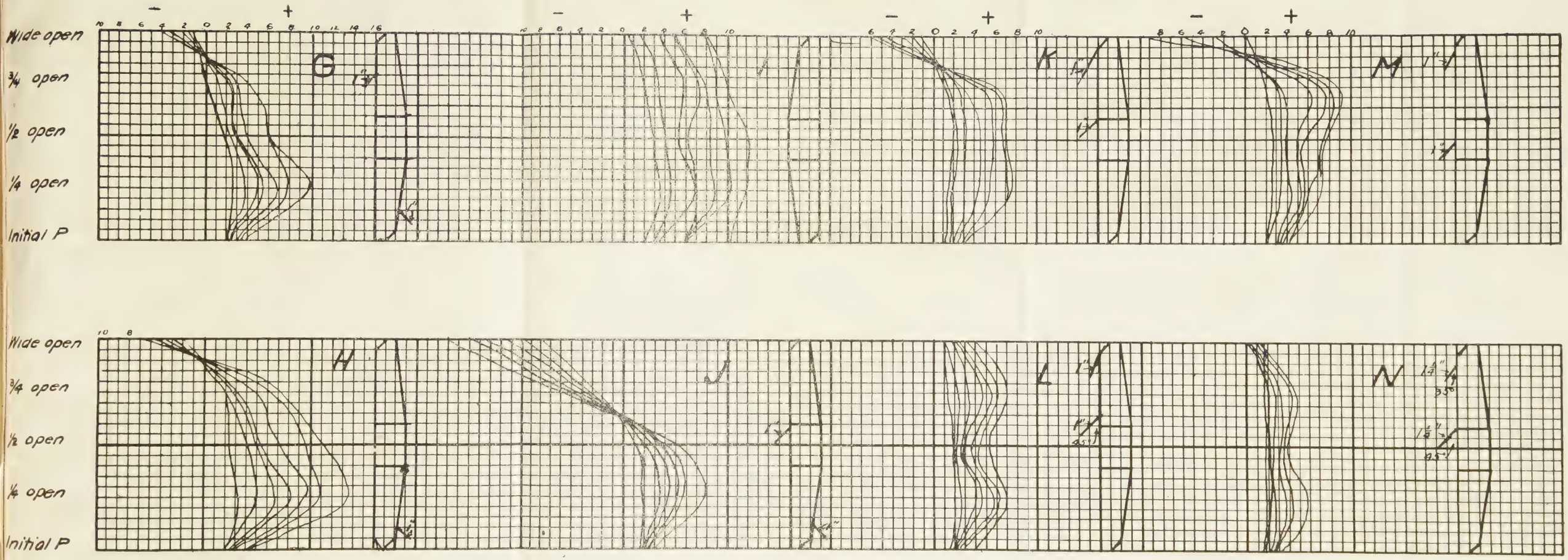


PLATE IIIb.

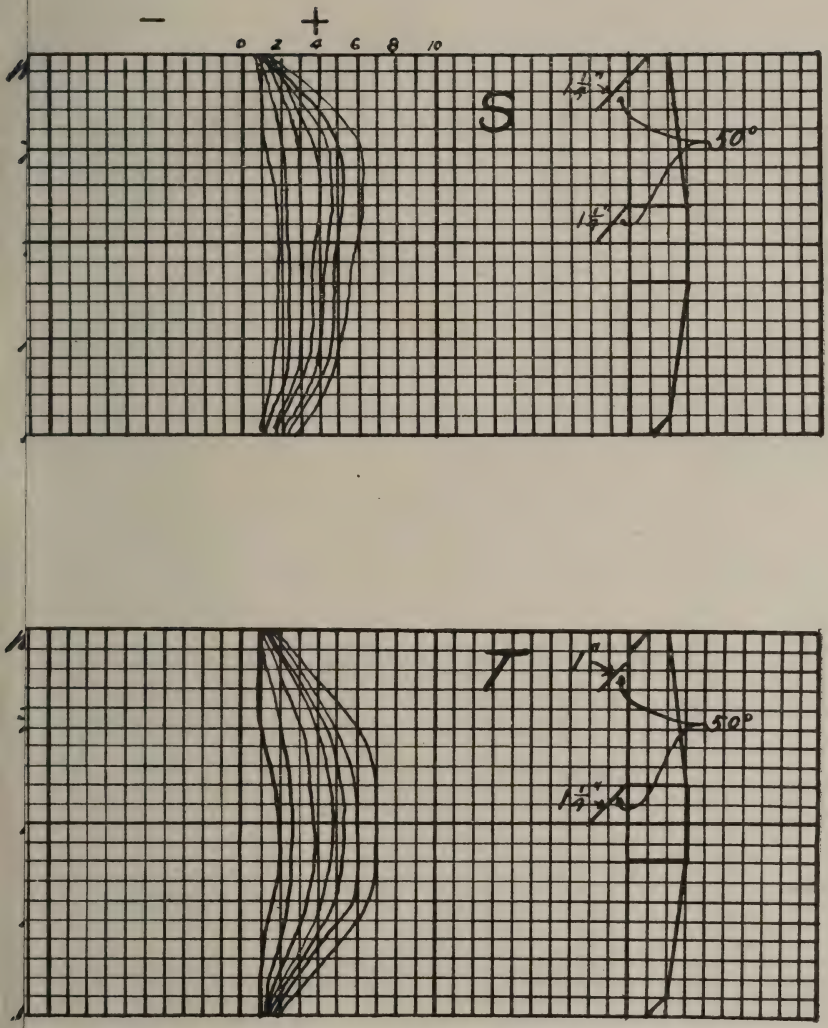
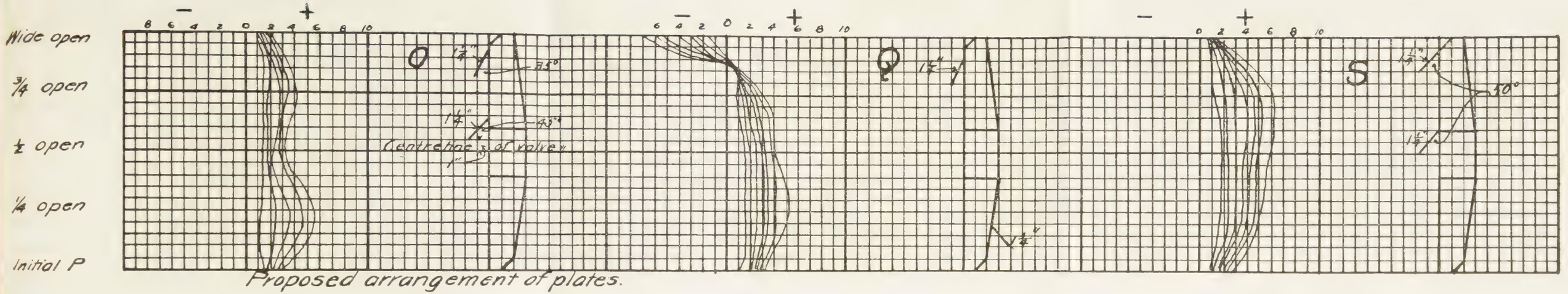


PLATE III c.



Proposed arrangement of plates.

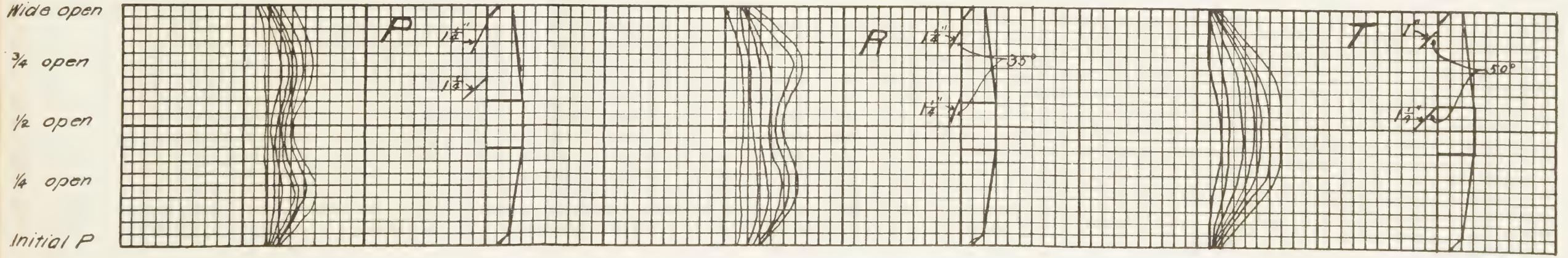


PLATE III c.

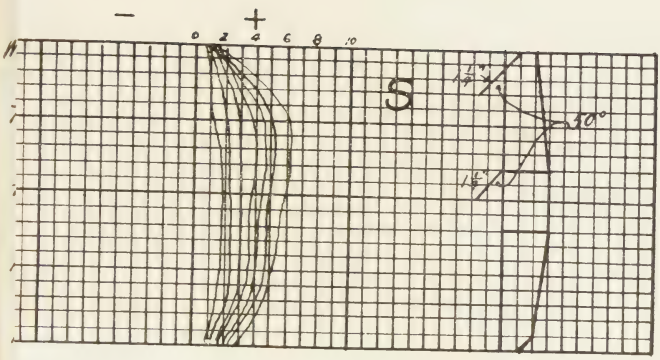
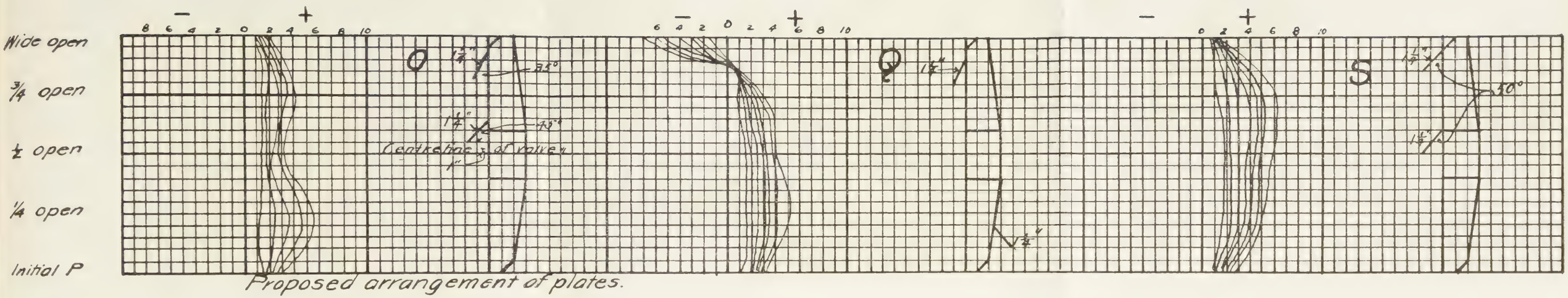


PLATE III c.

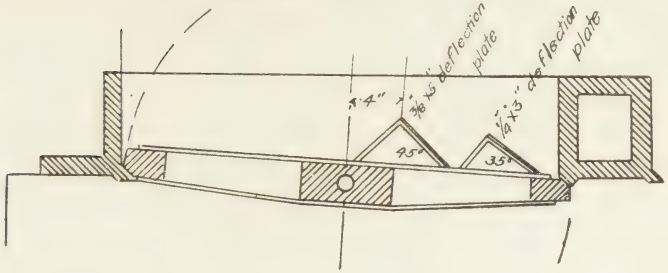


Proposed arrangement of plates.

PLATE III c.

Fig. C.

Section through upper bank wall valves as altered June 1912



Power required to operate middle valve. Up stream bank wall set.

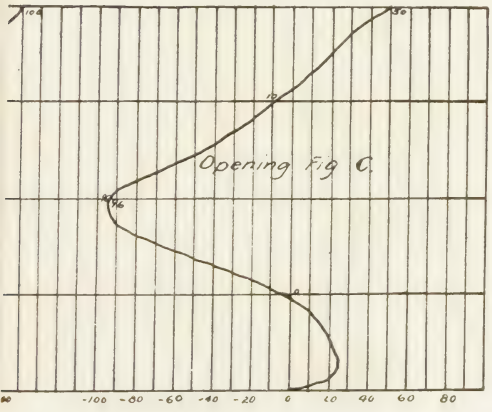
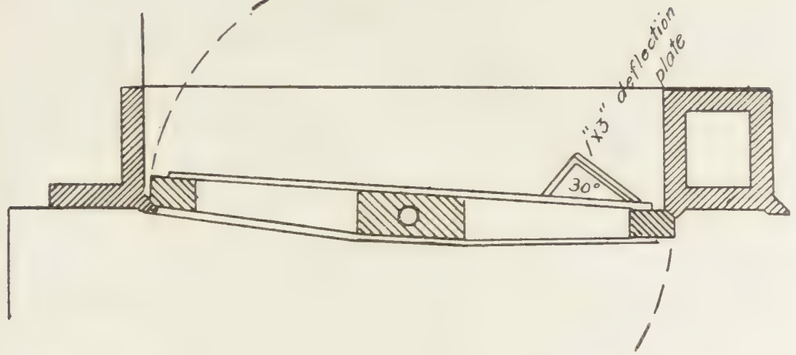


PLATE IV.
VALVE TESTS, LOCK 14.

Fig A

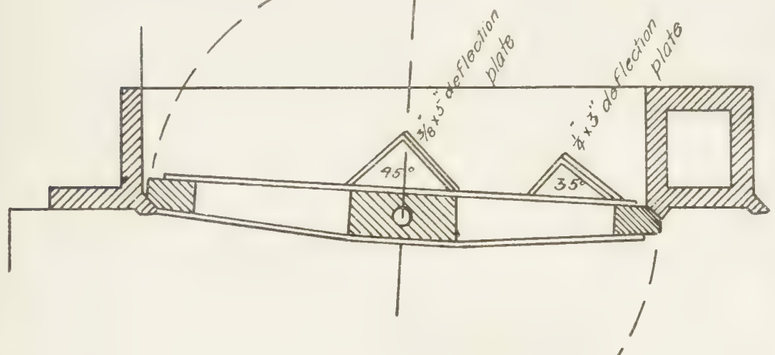
Section through all valves as originally built



Power required to operate upstream valves Lower bank wall set.

Fig. B

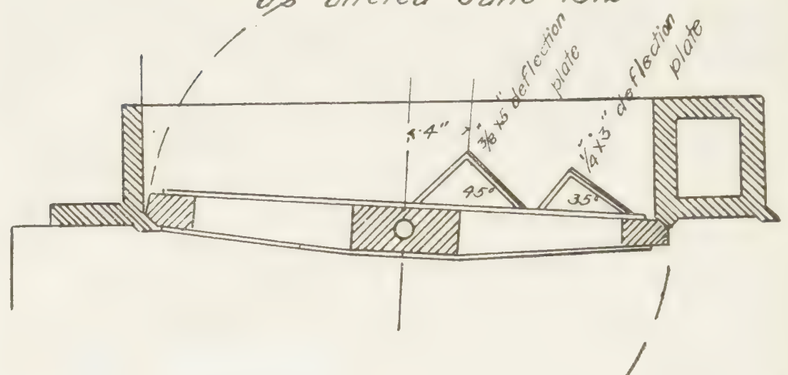
Section through lower valves and upper river wall valves as altered June 1912



Power required to operate middle valve Upstream river wall set.

Fig. C.

Section through upper bank wall valves as altered June 1912



Power required to operate middle valve. Up stream bank wall set.

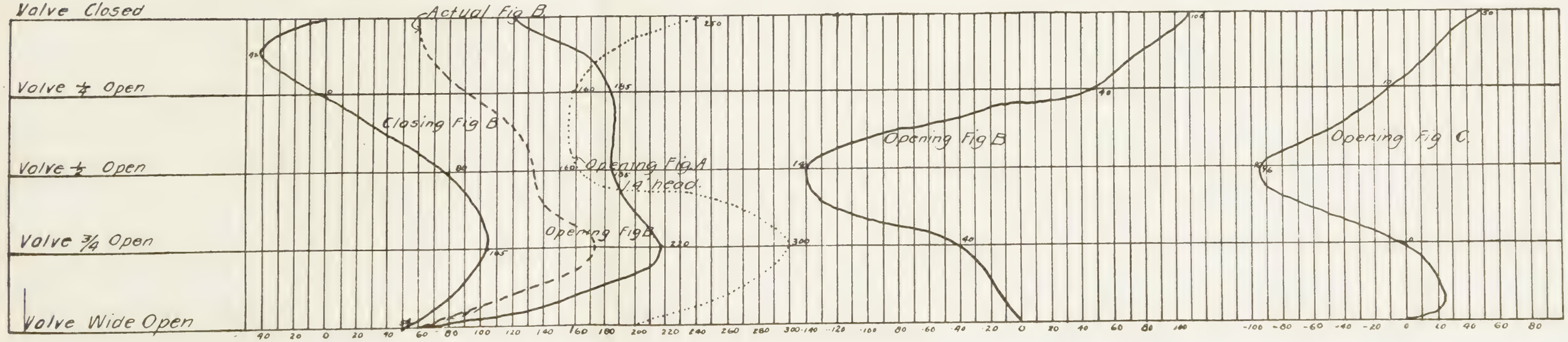


PLATE IV.
VALVE TESTS, LOCK 14.

It will be observed that the tests "N" and "O" gave similar results, and to enable a choice to be made between the two arrangements of deflection surfaces, in June, 1912, further tests were made at Lock 14 (14′ lift). The results are shown by the curves of Plate IV. On the first diagram, the dotted line shows the force applied to a 10′ lever to permit the opening under 14′ head of one lower valve, as originally built. The two full curves show the pressure necessary to open and to resist the closing of the same valve under the same head after the second deflection plate (shown in Fig. "B") has been added, and the heavy broken line indicates the actual resistance to opening due to water pressure only, frictional resistance being eliminated.

Comparison of the curves of the first diagram of Plate IV for a lower valve, with those for an upper valve (Figs. B and C, same plate) is interesting. The difference in these curves is due to the absence of backwater in the upper culvert, and shows that the sheet of water entering when the valve is "cracked" is deflected free of the in-turning half of the leaf by the second deflection plate, which causes the valve to slam open. The area of a single valve is so small, compared with the sectional area of the culvert, that the velocity through the valve opening is not accelerated by any draft-tube effect in the culvert as when the three valves are in service and the culvert is full of water. The head under which the tests at Lock 14 were made averaged 6 feet.

As a result of all the tests, it was decided to add to all butterfly valves then in service, in addition to the 3″ deflection plate originally provided, a second plate 5″ wide, secured to the back of each valve, with horizontal angle of 45° between them, and with the outer edge 4″ upstream from the center line of the back plate, as shown on Fig. "C" of Plate IV, thus conforming to the arrangement employed in Test "O" of Plate IIIc. The valves of Lock 16 (21′ lift), which had not yet been built, were designed to embody the same arrangement for deflection surfaces. This additional deflection plate, combined with the partitions between valves previously described, has made it possible for one operator to open to normal position the three valves of any set simultaneously and under full heads up to 21 feet.

The preceding account is derived from the records of the U. S. Engineer Office at Mobile. The experiments described are those referred to by Assistant Engineer Mower on pp. 320 and 321 of Professional Memoirs, No. 33.

The Development of Field Fortification in the Civil War.

BY

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In the preparation of this paper the writers will try to confine themselves to the discussion of field fortifications in their purity as they conceive the term to imply; that is, the use of them by two armies actually in the field. As far as extended investigation can show the birth of this idea occurred in our own Civil War, and the idea was fostered, developed and improved upon to such an extent that the later campaigns of this tremendous struggle present a phase of the use of field fortifications as fine as any examples to be found even in most recent times.

The history of the use of fortifications for the protection of restricted localities and vast regions of country is as old as the history of war itself. Homer, in his "Iliad," gives us an early picture of fortifications of a limited area for a city. Here these works of a purely defensive character served their purpose so well that it required the craftiness of a Ulysses, with his wooden horse, to accomplish that which the greatest Grecian warriors of the time had failed to do by direct assault. The great wall of China and the Roman wall across the north of England, the former to keep out the Tartar horsemen, and the latter the Picts and Scots, furnish us with fine examples of the use of purely defensive works on a large scale. The sieges of Sebastopol and Plevna, Vicksburg in our own Civil War, and of Port Arthur in more recent times furnish examples of the use of permanent and semi-permanent works, and of the proper methods in their attack and reduction by systematic siege operations. However, in all these cases, one of

the contesting armies was in a fortified place or one that had been strongly fortified as soon as it was occupied.

The development and use of field fortifications proper during the Civil War seem to have been due as much to the appreciation by the private soldier of their value and efficacy as to study and knowledge of their use by the higher officers. The education and intuition of the American soldier played an important part in the development under consideration. To him self-preservation was the first law of nature, and he was not long in appreciating that his chances for life were far better if, when the bullets were flying thick, he put a log in front of him or scooped out a hollow in the ground from which to fire. Hand in hand with this necessity for preservation went the inventive genius of the American, and the combination of the two developed a skill and rapidity in intrenching which was the marvel of most European observers of the time. It is doubtful if the value of field, or hasty intrenchments as they were then universally styled, was fully appreciated during the early months of the war. The troops of the west, or "Dirt Diggers" as they were derisively called, were preeminent in the use of fortifications of all kinds, but it was not until after the battle of Shiloh that they began to employ them extensively. Had their value been then appreciated it seems almost certain that at least a little work would have been undertaken on that field.

From this point on it is proposed to consider certain campaigns and battles at different periods of the war and to show to what extent field works were employed, how they served their purpose, what advantage was gained by their use, and, when they were not used, what results could have been expected from their use; and also lessons drawn from this which may be of value in a critical study of the subject.

The Confederate forces early realized the value of defensive works, and before the end of May, 1861, had made a substantial beginning on the line which ultimately extended from the Atlantic via Cumberland Gap—Bowling Green to the Mississippi. During this month works were erected at Centerville covering a front of 6,000 yards. As early as May 10 a battery had been erected by the Virginia troops covering the York River at Gloucester Point, which mounted at least one 8-inch gun. The work at Sewells Point, which covered the approaches to Norfolk, was so far completed on May 27 that it was able to fire upon Butler's expedition to Newport News. A battery at Pig Point, opposite Newport News, was

likewise completed during the month. May 30, the Confederate battery at Aquia Creek was shelled by three Federal gunboats. Batteries were also erected during the month covering the Potomac in the vicinity of Occoquan Creek, at Freestone Point, and at Fookes Landing near the mouth of Potomac Creek.

The first use of field fortifications prepared by the troops on the defensive in the field was at Big Bethel, Va., June 10, 1861, when the Confederate troops commanded by Col. (later Maj. Gen.) J. B. Magruder, succeeded in repulsing a superior force sent from Newport News for the purpose of driving them from the vicinity. The report of Col. (afterwards Lieut. Gen.) D. H. Hill, First North Carolina Volunteers, on the engagement is unusually detailed. This regiment arrived at Big Bethel on the 7th and at once began the organization of the sector assigned it and the preparation of a work for the artillery. General Hill states (Off rec. U. & C., Ser. No. 2, p. 93): "We had only twenty-five spades, six axes and three picks, but these were busily plied all day and night of the 7th and all day on the 8th. . . . On Sunday, the 9th, a fresh supply of tools enabled us to put more men to work, and when not engaged in religious duties, the men worked vigorously on the intrenchments."

At 9 a. m. on the 10th the National troops appeared in front of the position, but were driven off and pursued all of the way to Newport News. General Hill adds: "After the battle they (the First North Carolina Volunteers) shook hands affectionately with their spades, calling them 'clever fellows and good friends.'" Maj. G. W. Randolph, of the Confederate Artillery, states in his report (*id.*, p. 101): "The untiring industry of his regiment (D. H. Hill's First North Carolina) in intrenching our position enabled us to defeat the enemy with a nominal loss on our side."

The Confederate losses were one killed and seven wounded. Butler's report stated the Federal losses to be eighteen killed and fifty-three wounded.

It is worthy of more than passing notice that notwithstanding this and many other instances which pointed strongly to the advantages of defensive organizations, their use as a rule was not resorted to until nearly three years later.

For the purpose of opposing the advance of the National forces commanded by McClellan in West Virginia, Brig. Gen. R. S. Garnett, C. S. A., on June 15, 1861, occupied the passes over Rich and Laurel mountains to the west of Beverly and Leafsville and

covering the roads to Buckhannon and Philippi. The defensive organization contemplated must have been very elaborate, since Garnett reported to the War Department ten days after his arrival that the defenses would be completed in about a week. The work on the two positions continued until July 10, when McClellan's troops appeared before them. The Laurel Mountain position, which was about 12 miles north of that on Rich Mountain, was contained by a portion of the National forces advancing from Philippi, while the larger portion of the command under McClellan operated against the works covering the Rich Mountain pass. This position was, in general, a good one, but occupied a front of only about 1,300 yards, and was situated on the western slope of the mountain 1½ miles from the summit. During the night of the 10th-11th Rosecrans, with about 2,000 men, after a march of about 6 miles, succeeded in moving around the position and reached the summit, where he defeated a small force sent to oppose him, suffering the very insignificant loss of twelve killed and fifty-nine wounded. Both positions being thus turned, the Confederates withdrew. This engagement furnishes an excellent illustration of the futility of expending much effort on works which can not meet the conditions for which they have been constructed—twenty-five days had been spent in preparing the position from which the defenders were compelled to retire in a few hours without delaying their opponents or inflicting any commensurate loss on them.

Shiloh furnished an imposing example of the "what might have been" in the use of field works. General Grant's Army of the Tennessee was in the vicinity of Savannah, Pittsburg and Crumps Landing on the 17th of March, 1862, the day when General Grant returned to it after his suspension from command by General Halleck. The battle of Shiloh was fought on the 6th-7th of April, twenty-one days later. Returns for April 4th-5th show General Grant's strength to have been approximately 42,000. At this time General Johnston was known to be at Corinth, with the bulk of the Confederate forces in the vicinity, estimated at from 60,000 to 80,000. General Grant was awaiting the arrival of General Buell with the Army of the Ohio before beginning offensive operations. Despite all these conditions absolutely nothing was done towards strengthening the Federal position, although McPherson, Grant's Chief Engineer Officer, had reconnoitered a line for defense. However, it was found that the use of this line would necessitate the removal of the camps of some of the brigades, so nothing was done.

Both Generals Grant and Sherman evidently realized in later years that the Federals were at fault at this time for not preparing their position. General Grant states in his Memoirs, Vol. I, page 332: ". . . I expected to take the initiative by marching on Corinth, and had no expectation of needing fortifications, . . . the fact is, I regarded the campaign we were engaged in as an offensive one and had no idea that the enemy would leave strong intrenchments to take the initiative when he knew that he would be attacked where he was if he remained." Again (page 358) "Under all these circumstances I concluded that drill and discipline were worth more to our men than fortifications." General Sherman says in his Memoirs, Vol. I, page 229, "We did not fortify our camps against an attack because we had no orders to do so, and because such a course would have made our raw men timid. . . . At a later period of the war we could have rendered this position impregnable in one night. . . ." Then he adds as an afterthought, "And it may be it is well we did not." Again on page 247, "We had no intrenchments of any sort, on the theory that as soon as Buell arrived we would march to Corinth to attack the enemy." These excuses for the lack of fieldworks, coming from Grant and Sherman, who later became past masters in the art of using hasty intrenchments, show that they fully appreciated the fault at Shiloh, and were looking for the best reasons to justify their actions. Imagine, if you can, either Grant or Sherman saying in '64 that they did not use fortifications because they were on, or about to take up, the offensive.

A glance at a map of the position occupied by the Federal troops on April 6 will instantly give us an idea of what might have been accomplished. The position extended generally along high ground from Owl Creek on the right to Lick Creek and the Tennessee River on the left, a distance of about 6,000 yards. General Grant had, to occupy this line, from 30,000 to 35,000 rifles, or five to six men per yard of front. That Johnston's army, if he had attacked this position, would have been cut to pieces, can hardly be questioned. What the turn of events might have been had Johnston not attacked is a matter of speculation; perhaps, even as General Sherman said, "It may be it is well we did not intrench."

In sharp contrast to the lack of field works at Shiloh is the advance of Halleck on Corinth, where nearly every foot of ground was fortified and over one month was required to march 20 miles. With the change of commanders the pendulum swung to the other

extreme. From these two extreme examples we see a gradual increase in the knowledge of when and how to use field works, with both sides profiting by this knowledge, especially in the western theater.

In the eastern theater of operations we find the first extensive use of field works in the Peninsular campaign of the Army of the Potomac in the spring and summer of 1862. In the operations covering the period from the advance from Fort Monroe, April 4, 1862, to July 3, 1862, when the Army of the Potomac found itself in the intrenched camp at Harrisons Landing, we find two distinct periods. In each of these the Union and Confederate armies were respectively on offensive and defensive, and then on defensive and offensive. In each period we find defensive fieldworks playing the most important part, in the first delaying and paralyzing the Union advance and finally aborting the entire campaign; in the second thwarting complete victories on the part of the Confederates, and saving the Army of the Potomac from complete destruction. In the first period their effect was largely moral, and bore most fruit in their effect directly upon the character of the Union commander. In the second the effect was largely physical, and showed itself almost entirely on the fighting personnel of the Confederate Army; for example, the action at Mechanicsville on the 26th of June, when the Confederate losses were nearly one-third the number of all the Union troops engaged.

At the time of the Union advance up the Peninsula, Gen. J. B. Magruder, in command of the Confederates in the vicinity of Yorktown, profiting by the experiences gained at Big Bethel, made extensive use of field fortifications. Realizing that a Union advance on Richmond via Yorktown was extremely probable he early took steps to oppose such advance, erecting works along the line of the Warwick River and across the Peninsula near Williamsburg. The Yorktown lines, at which the advance of McClellan was stopped for one month, consisted of works of a more or less semi-permanent nature, constructed with the assistance of slave labor. They consisted of the extensive works around Yorktown itself, from there extending southeast to the line of the Warwick River. This line embraced five dams to deepen the water in the creek, as well as earthworks and gun-pits covering the crossings and the dams. These groups of fieldworks were connected by rifle trenches for nearly the entire distance from Winns Mill to Lees Mills. Abattis was constructed along the entire front of these infantry trenches.

From Lees Mills to Southells Landing other works were constructed, but owing to the especially difficult nature of the country nearer the James River these works had little bearing on the Yorktown affair.

The entire length of line occupied by General Magruder, exclusive of the works of Yorktown, was about 11 miles. He had, at the beginning of the siege, between 11,000 and 12,000 troops, much less than 1,000 men per mile, since about 5,000 troops were assigned to the immediate works at Yorktown. By the 11th of April his forces had been increased to 35,000, and by the 17th about 53,000 men were along the Warwick lines.

General McClellan's force, for the advance on Yorktown, was about 85,000 men, as reported by him on the 4th and 7th of April, and up to the 11th of April his command was nearly seven times as large as the forces under Magruder. Reconnaissances developed the extent of the Confederate fortifications, but not the number of troops holding them. General McClellan had been present at the Siege of Sebastopol, and had there learned the power of fortifications against troops advancing in the open. No determined assaults were made on the Warwick lines. Siege operations were undertaken. At the end of one month, just before the immense Union siege batteries were ready to open fire, the Confederates withdrew. Can it, from these facts, be doubted that it was the moral effect of these works on the mind of the Federal commander which delayed the advance of the Union Army for one precious month, during which time the defenses of the Confederate capital were greatly strengthened? Yorktown was won without great loss of life, but what the month's delay lost to the Union cause we are unable to calculate. Certain it is that time was gained to collect an army at Richmond, and to improve its fortifications and, so leading Confederate generals stated, Richmond was saved.

Following the evacuation of Yorktown on the 4th of May the Union troops took up the pursuit. On the 5th the battle of Williamsburg was fought. The Peninsula at this point is between 6 and 7 miles wide between the James and York rivers; a line of forts had been constructed across this narrow strip. The Confederate movements and dispositions indicated an intention to act defensively in these works. On the morning of May 5 Hooker's division, which was on the left of the Federal advance, attacked the Confederate works in the immediate front of Williamsburg without waiting for the troops on their right to deploy. The attack

was promptly checked, and the Confederates in Hooker's front, observing his isolated condition, took the offensive against his left. The division was driven some distance to the rear, losing many men and a part of the guns with which the attack had been opened. At 3 p. m. the condition of the division was such that it was saved from annihilation only by the opportune arrival of Kearny's division. The use made by the Confederates of fieldworks on this occasion was very skillful, and considering the quality of the troops at that time of the war, quite remarkable, since they without hesitation left their works for the purpose of maneuvering in the open when it appeared to their advantage to do so.

From the 25th of May, when Keyes was ordered with his Fourth Corps to take and hold a position near Seven Pines road forks, until the 26th of June, the day of the Mechanicsville fight, both the Union and Confederate armies were engaged more or less continuously in intrenching their lines south of the Chickahominy. The Union lines in this section, when finally completed, were over 3 miles long, reaching from the White Oak Swamp on the left to the Chickahominy, near Duane's foot bridge, on the right. They embraced six forts, mounting in all forty guns, connected by a continuous line of rifle pits. The latter had a parapet well raised above the surface of the ground, 3 or 4 feet thick at the top, and had a ditch in front. The redoubts had strong profiles, being 10 to 12 feet thick at the top, with a deep ditch in front and magazines in rear. Opposed to this Union line were the Confederate works, reaching from the James to the Chickahominy and of even greater strength, since they embraced a part of the defenses of Richmond.

From the Union viewpoint, the results obtained from their works were entirely incommensurate with the amount of labor expended. On the other hand, the service of their works to the Confederates was important. Behind their lines, on the 27th of June, General Magruder, with his paltry 17,000 men, "kept up such a clatter" as completely to mislead the Union commanders, and to hold off from assault fully 65,000, while Lee with the main strength of the Army of Northern Virginia was hammering Porter back south of the Chickahominy. On this day the initiative passed to the side of the Confederacy, and defensive fieldworks began their important task of saving the Army of the Potomac.

In pursuance of the general plan to place his troops in intrenchments around Richmond and to await General McDowell from the north, General McClellan had intrusted to General Porter the safety

of the Union right, and the line of communications north of the Chickahominy. Following out these orders, a strong line of works had been placed along the high ground just east of and commanding Beaver Dam Creek. This line, about 3,500 yards long, was strengthened by rifle pits and two fieldworks for artillery, the two latter covering the crossings over Beaver Dam Creek at Ellersons Mill and on the road from Mechanicsville to Bethesda Church. The rifle pits were generally shallow trenches with a parapet revetted with logs. Epaulements were constructed for the artillery. In the fight of June 26, eleven regiments, aggregating about 5,000 men, held this line of works—a little less than one and one-half men per yard of front. The attacking force consisted of thirty regiments and seven batteries, about 16,000 men in all. How well the intrenchments accomplished their purpose is evidenced by the fact that the Union lines held out during the entire afternoon. The approximate losses were: Union, 360; Confederate, 1,760.

During the night of June 26-27, General McClellan, owing to the reports of Jackson's advance threatening the Union right, directed Porter's command to withdraw to a position in the vicinity of Gaines Mill. This position had been reconnoitered on the 26th by General Barnard, Chief Engineer of the Army of the Potomac.

The battle fought at that place marks an important epoch in the Federal employment of field fortifications, since it is thought that here, for the first time in the history of the Army of the Potomac, is to be found the use of hasty intrenchments constructed on the spur of the moment by troops just deployed and awaiting the hostile advance. Up to this time, it will be noticed, all works discussed had been prepared more or less at leisure, at some time previous to the day when they were to be used, and had been rather elaborate.

The following extracts from the reports of regimental commanders, during the action, are but samples of similar reports from all:

Lieutenant Colonel Campbell, Eighty-third Pennsylvania Volunteers (Off. rec. U. & C., Ser. No. 13, p. 344): ". . . the regiment stacked arms, . . . threw up a temporary breastwork of all the felled timber and rubbish along the line." Major Tilton, Twenty-second Massachusetts (*id.*, p. 301), "I still more strengthened our position by felling great pines and constructing of them and smaller trees a barricade." Colonel Roberts, Second Maine (*id.*, p. 296), "Having the advantage of a rail fence, I ordered my boys to make with their knapsacks a barricade, which

they did in a very short time." Maj. Gen. Fitzjohn Porter (B. & L., Vol. II, Part 1, p. 335), "The axes did not arrive till near dark, and were useless; but with the few obtained early in the day from the artillery, and in the little time at command, trees were felled along a small portion of our front, and barriers were erected, which were filled in with rails and knapsacks."

The line thus strengthened was about 3,500 yards in length, extending from a point about 1,000 yards from Duane's footbridge in a semi-circle towards the northeast, and crossing the road from Old Cold Harbor to Despatch Station, about 1,300 yards directly south of the former place. To occupy this line Porter had about 20,000 men or six men to the yard of front. Averaging the Confederate regiments, at this time at about 500 men, Lee attacked with about 65,000. The first attack began about 12.30 p. m., and it was not till nightfall that the Union lines finally gave way, and Porter's troops began their withdrawal south of Chickahominy.

In addition to presenting one of the first instances of the use of hasty intrenchments, this battle has another interesting point for our consideration. All of the reports show that at this time the axe was the tool for hasty work. This development was but natural and shows the ability of the American soldier to adapt himself to circumstances. Timber, especially pine and evergreen, was very abundant in the country through which the army was operating. The average penetration of the small arms was about 6 inches of pine at 500 yards, and trees and logs of medium size furnished sufficient cover. This breastwork quickly thrown up served its purpose, at first, and later, when it became possible to improve the works, it served as an excellent revetment for earth taken from the trench and piled up in front as a parapet.

The engagements on June 28 and 29 are of little importance to our discussion, since, where fieldworks were used, they were ones which had previously been prepared and do not show any development in the use of hasty fortification. The engagement of the 30th of June at Frazier's farm is, however, of interest. The troops engaged were some of those who served at Gaines Mill and made such good use of quickly prepared field works. However, even by these troops, the lesson was not yet learned. General McCall, whose division stood the brunt of the fighting on this day, reporting on the battle, says (Off. rec. U. & C., Ser. No. 13, p. 389): "Reached the point where the Turkey Bridge Road turns off from the New Market Road at about 7 a. m. . . . (About 9 a. m.) I examined

the character of the country. . . . The country on my new front was open, embracing a large farm. . . . The open front was about 800 yards. . . . My disposition having been made, I calmly awaited the approach of the enemy. About 2.30 o'clock my pickets were driven in. . . ." General Truman, in his report on the battle, states (*id.*, p. 403), "On the extreme left a small log farmhouse (Whitlock's) had hastily been prepared for defense by piling rails and logs so as to shelter a part of the Twelfth Regiment. . . ." Lieutenant Colonel Taggert, commanding this regiment, says (*id.*, p. 428), "six companies . . . were posted in two log huts and a breastwork of rails temporarily thrown up about 200 yards in advance. . . . One column of the enemy attempted to surround the men in the breastwork and log huts . . . Seeing that they were about to be surrounded the men retired. . . ." In this engagement we see a position occupied several hours before the enemy appeared, and yet, apparently from failure to appreciate the value of fortifications, no fieldworks were prepared. We note one regiment preparing its position but unable to hold it owing to the fact that regiments on its flanks were driven back, exposing it to capture. Following the battle of June 30, we find the battle of Malvern Hill (July 1) fought entirely without intrenchments, though the position was occupied some hours previous to the approach of the enemy. General McClellan evidently considered the matter of preparing a position since, in reporting on the battle, he said that he depended on the great natural strength of the position and the superior position of the artillery to enable him to hold it. From Malvern Hill the army retired into its position around Harrisons Landing. This position had been carefully reconnoitered by the engineers, and as soon as the troops reached it they were at once put to work fortifying it. The line, as finally occupied, consisted of a series of strong redoubts all connected by rifle pits. The latter, in the field through the open, consisted of infantry parapets of earth; through the woods they consisted of barricades of logs and timber strengthened by earth thrown up against it from the outside. General Lee in his report (Off. rec. U. & C., Ser. No. 13, p. 497), says of this position: "He immediately began to fortify his position, which was one of great natural strength, . . . It was deemed inexpedient to attack him, . . ." Here, in an intrenched camp the Peninsular campaign ended, the works around the army at Westover having much the same effect on the judgment of General

Lee as did the lines at Yorktown on General McClellan, though with results much less disastrous and far-reaching.

The defensive rôle of field fortification was so successfully carried out during this campaign that its value should by that time have been recognized by all. Such does not appear to have been the case with either the Army of Northern Virginia or of the Potomac. Though the teachings had been excellent, the lesson was still unlearned. The latter army, under Pope at the second battle of Bull Run, though on the defensive, made little or no use of hasty intrenchments. Later at Antietam, with the rôles reversed, Lee, strictly on the defensive, strengthened but a small part of his line. For Lee this battle evidently had its lesson, and the service of field-works in the Peninsula were perhaps again called to mind, since from this time forth, wherever the armies of Lee stood on the defensive, there they intrenched.

November 25, 1862, McLaws' division of Longstreet's corps occupied Fredericksburg, the opposite bank of the Rappahannock being held by Federal troops. Work was immediately begun on rifle pits along the right bank of the river, and buildings in the vicinity were prepared for defense. Communications between the town and the trenches along the river bank were constructed, all of the work being done at night and apparently without the knowledge of the Federal covering troops across the river. From these very light works, and without the aid of artillery, Barksdale's brigade succeeded in checking from daylight until nearly nightfall, December 11, all attempts of the National forces to throw a bridge, notwithstanding the fact that all of the National heavy ordnance on Stafford Heights within range was concentrated upon the town, and in addition six light 12-pounder batteries were withdrawn from their divisions and advanced to the northern bank of the river. The report of Brig. Gen. H. J. Hunt, Chief of Artillery, Army of the Potomac, states: "A severe cannonade was now opened on the cover which protected the enemy's sharpshooters . . . But their sharpshooters succeeded in stopping the work on the bridge." The crossing was forced by means of pontons about dusk, but the bridge was not thrown until evening. The delay caused by these insignificant works was of priceless value to the Confederates in their concentration and preparations for the battle which took place two days later. "These works consisted of a loop-holed blockhouse near the upper bridge, a rifle pit about 200 feet long and numerous cellars enclosed by stone walls."

(Report of Brig. Gen. D. P. Woodbury, Commanding Engineer Brigade, Army of the Potomac, 12th Dec., 1862.)

The main Confederate position extended along the heights from ¾ to 2 miles to the west of Fredericksburg, and was organized for defense generally in the conventional manner of the period. A striking innovation, however, appeared in the organization directly to the west of the town, where the Telegraph Road extends south for about 800 yards from the Plank Road, skirting the base of Maryes Heights, before turning west along the north bank of Hazel Run. This stretch of road was beyond the range of the heavy National ordnance on Stafford Heights. An excellent description of the position appears in the report of Brig. Gen. Joseph B. Kershaw, C. S. Army, whose brigade formed a part of the garrison. He states (Off. rec. U. & C., Ser. No. 31, p. 589): "The position was excellent. Maryes Hill, covered with our batteries—then occupied by the Washington Artillery, Col. (J. B.) Walton commanding—falls off abruptly toward Fredericksburg to a stone wall, which forms a terrace on the side of the hill and the outer margin of the Telegraph Road, which winds along the foot of the hill. The road is about some 25 feet wide, and is faced by a stone wall about 4 feet high on the city side. The road having been cut out of the side of the hill, in many places this last wall is not visible above the surface of the ground. The ground falls off rapidly to almost a level surface, which extends about 150 yards, then, with another abrupt fall of a few feet, to another plain which extends some 200 yards, and then falls off abruptly into a wide ravine, which extends along the whole front of the city and discharges into Hazel Run." Concerning the manner in which the position was held he adds: "I found, on my arrival, that Cobb's brigade, Colonel McMillan commanding, occupied our entire front, and that my troops could only get into position by doubling on them. This was accordingly done, and the formation along most of the line during the engagement was consequently four deep." It does not appear that this position was in any manner improved after its occupation. During the 13th of December about 30,000 Federal troops were moved at different times against this position. Six general assaults were repulsed. The losses of the Federal commands directed against the Maryes Heights position were 7,557. Cobb's and Kershaw's brigades, which held Telegraph Road, lost 614 killed and wounded, and Longstreet's entire corps but 1,519.

The position occupied by Barksdale's brigade on the 11th is a

good example of an advanced position, while that at the base of Maryes Heights was in the nature of an advanced post. Both show the limitations and possibilities of such organizations under certain conditions.

The capture of the Maryes Heights position by troops of the Sixth Corps commanded by Sedgwick on the 3rd of May following, shows, however, that such positions may not under all circumstances be appropriate for the demands of the situation. The Telegraph Road position offered little resistance to the National main attack. "Along the wall a hand-to-hand fight took place, and the bayonet and butt of the musket were freely used. The brilliant and successful charge occupied perhaps ten or fifteen minutes, and immediately after the wall was carried the enemy became panic-stricken" (Lieut. Col. H. W. Jackson, B. & L., Vol. III, p. 229).

In this case the attacking force consisted of about 7,500 men, and the garrison of the road position about 600, the proportion being 7½ to 1. At the first fight the numbers were respectively about 30,000 (including the Fifth Corps) and 3,000, the proportion being 10 to 1. It is not impossible that the operations in the vicinity of Chancellorsville had some influence upon the actions of the Confederates.

With the Army of the Potomac we find little use of fieldworks until 1864, when Grant came from the west to assume command in the east. However, one notable exception is the battle of Chancellorsville. This engagement has for us a very important bearing, since it was the first great battle of the war where both sides used field fortifications extensively. A study of the Rebellion Records shows that virtually three lines of intrenchments were prepared by Hooker's troops during these memorable five days. The first line of Hooker's was turned May 2 because Howard's right flank was in the air. During the night of May 2-3 the Union right in front of Stuart, who had relieved Hill when the latter was wounded, had been readjusted and prepared with breastworks and abattis. Again the location was faulty; the right wing was still in the air, of which fact Stuart promptly took advantage. During the night of May 3-4 a third line was intrenched, from the Rapidan on the right to the Rappahannock on the left. Both flanks were secured and breastworks of earth and logs, strengthened by abattis, were erected over the entire front. This line, Alexander says, was

"probably the strongest field entrenchments ever built in Virginia." Hooker made no use of them.

Field fortifications served the Confederates better than they did the Federals. The first line taken up by Anderson and reinforced by McLaws may have caused Hooker's great blunder, which was to fall back to Chancellorsville on May 1, after nearly reaching Banks Ford and open country in which he could maneuver. The second line of works confronted the Union lines from the vicinity of Catherine Furnace to the Rappahannock. These fortifications eminently served their purpose, since all day May 2, while Jackson was making his flank march to the Union right, there were but about 16,000 Confederate troops in this section opposed to nearly 50,000 Federals. Again on May 3 and 4, when Lee with the bulk of his forces turned back against Sedgwick, near Salem Church, 20,000 men in this line contained Hooker with 60,000 Union troops.

In addition to the works used by Lee's main army near Chancellorsville, the troops under Early at Fredericksburg, opposing Sedgwick, made excellent use of the old lines prepared during the days of the Fredericksburg campaign.

Chancellorsville has, for the student of field fortification, one important lesson; namely, the strength of fieldworks is measured not so much by the impregnability of its front as by the security of its flanks. Had Howard's exposed right flank been refused to the strong position along Hunting Run, instead of being extended in the air, along the Old Wilderness Turnpike, the history of this battle might furnish far different points of interest for the military student.

Hooker's use of fieldworks in this campaign can not be recommended to a student of military history as a model. The first line, as intrenched, was of no service owing to its exposed right flank. The second was of some assistance in stopping the advance of Stuart. By not remaining in his third line on the 6th of May it is said that General Hooker made the mistake of his life. As already stated, General Alexander says "this line, the flanks of which were impregnable, was the strongest field fortification ever built in Virginia." That Lee, with 35,000 troops, contemplated attacking Hooker's 90,000 behind these works on the 6th is certain from his own statements. No doubt only Hooker's withdrawal saved Lee from a terrible repulse. As shown by his message to Sedgwick, "Am in intrenchments hoping to be attacked," Hooker

was attempting to combine defensive tactics with offensive strategy. The employment of fieldworks for such a purpose is justified, provided the commander is a great enough master of the art of war to carry the action through. History shows us but few examples of leaders other than Napoleon who were able to combine the two. Whether Hooker might have made such use of his magnificent third line can never be decided, since he lost confidence in himself and withdrew before the attack came.

Let us turn now for a review of the work of the armies in the western theater. From the beginning the Confederates, in almost all the important engagements in the west, were on the defensive, and the leaders early recognizing the value of fieldworks, we are not surprised to find them employed much more extensively than in the east. Since the reduction or capture of the semi-permanent works generally required siege operations, the Union troops learned much earlier in the war the true value of fortifications of all kinds. That the use of fieldworks was not confined entirely to defensive tactics is illustrated by the battle of Murfreesboro. In this engagement the Union left and the Confederate right and center were intrenched. Bragg, taking the offensive, attacked the unintrenched portion of the Union line, and rolled it back, the disaster continuing from the right towards the left until it hit the brigade of General Cruft, covered only by a rail fence. Here it was that, with the assistance of Hazen's brigade, the victorious Confederates were checked, and as it did later at Chickamauga, so here the strengthened portion of the Union line saved the day and turned what might well have been an overwhelming defeat into almost a drawn battle.

Passing now to the battles around Chattanooga we find some excellent examples of the use of field fortifications. Shortly after his evacuation of Chattanooga, General Bragg was largely reinforced and determined on an assumption of the offensive. His orders for this were issued September 18. At this time the Army of the Cumberland was badly dispersed on the west bank of Chickamauga Creek. Rosecrans, divining that Bragg's efforts would be to cut him off from Chattanooga, at once ordered a concentration on his own left flank. This was effected in time to meet the Confederate attack of the 19th. With this affair we are little concerned since fieldworks bore no part. The plan to cut the Federals off from Chattanooga via Rossville and McFarlands Gap had not succeeded, but was to be resumed early September 20. As Gen-

eral Bragg, in his report (Off. rec. U. & C., Ser. No. 51, p. 33), says: "Lieutenant General Polk was ordered to assail the enemy on our extreme right at day-dawn on the 20th, . . . My orders were renewed, and the General was urged to their prompt execution, the more important as the ear was saluted throughout the night with the sounds of the axe and falling timber as the enemy industriously labored to strengthen his positions by hastily constructed barricades and breastworks."

Rosecrans says in his report (Off. rec. U. & C., Ser. No. 50, p. 57), that he realized "that the battle on the next day (20th) must be for the safety of the army and the possession of Chattanooga." His orders for the 20th bear out this statement. The point of interest for the purposes of this paper, in this day's fight, is the action of the Fourteenth Corps under Thomas on the left flank of the Union Army, consisting of the divisions of Baird, Palmer, Johnson and Reynolds. General Baird, in his report (*id.*, p. 277), states: "During the interval between daylight and the first attack our men worked vigorously and covered themselves with a hastily constructed breastwork of logs and rails, which proved of vast service to them during the day." General Reynolds states (*id.*, p. 441), "During the early morning of the 20th temporary breastworks were erected of such material as could be found at hand, and were of great benefit." General Baird was on the left of the intrenched section of the Union line and Reynolds on the right. Reports of Palmer and Johnson are similar in regard to the trenches constructed to the two reports quoted. Next on the right of Reynolds came in order the divisions of Brannan, Negley, Wood, Davis and Sheridan with Van Cleve in reserve. None of the six last named intrenched their lines.

Pursuant to the plan of General Bragg, the first attack of the 20th fell upon the Union left hitting the intrenched lines of Thomas about 9 a. m. This part of of the line bore the attacks till about noon, when, in the successive order of battle adopted by Bragg, Longstreet advanced. His advance struck an interval in the line near Brannan's position, temporarily without troops. The attack of Longstreet threw into confusion the unintrenched divisions on the Federal right, and three of these (lacking one brigade) were driven entirely off the field to return only in time to cover the withdrawal of the Federal Army late in the evening. The divisions of Brannan and Wood, neither of which had intrenched, were pushed back, but in a lull in the battle they found time to reform

on Snodgrass Hill, and with Stedman's division of Granger's reserve corps, did excellent service in protecting Thomas's right during the remainder of this eventful day. Stuart's division of Longstreet's corps in its advance came upon the intrenched line of Reynolds and was stopped. This action of Reynold's division was but an example of the work of the other three divisions under Thomas, whose infantry and artillery "standing like a rock" behind their temporary breastworks throughout the entire day, withstood the individual and combined assaults of six Confederate divisions. From noon until dark these divisions, covered on their flanks by the troops of Granger, Wood and Brannan, maintained themselves against the continuous assaults of Bragg's 70,000 men. So well had the intrenched divisions done their work that when at 3 p. m. Longstreet requested of Bragg troops from the Confed erate right in order to push home the advantage he had gained over the Union right, Bragg sent back the message that these troops had been beaten back so badly that they could be of no service to Longstreet.

Here again, as at Murfreesboro, we find the strengthened position of the line maintaining itself undefeated, though the other parts of the line were beaten back around it as a pivot.

Following this battle, Rosecrans retired to Chattanooga and intrenched himself from the Tennessee River on the north around to the river again on the south. Bragg, attacking in the open at Chickamauga and failing, now drew a line of works on a larger circumference around Rosecrans, sat down behind them, and by starvation nearly accomplished that which he had failed to do by direct assault.

The Battle of Chattanooga, wherein Grant, who had been sent to command at that point, accomplished the relief of the Army of the Cumberland, furnishes us several excellent examples of the progress in use of field fortifications. The lines of investment of Bragg extended along the summit of Missionary Ridge from Tunnel Hill and the Tennessee River on the north to Lookout Mountain on the south. The general plan of battle adopted by Grant included an assault on the extreme Confederate right by the troops under Sherman. These latter troops were fresh from their experiences with earthworks around Vicksburg, and their training is at once evident in their use of these at this time on the offensive. During the night of November 23-24 Sherman, by means of pontons, crossed two divisions of about 8,000 men over the Tennessee.

The first work of these troops was to form a bridge head to cover the throwing of the ponton bridge for the remainder of Sherman's troops. All reports of brigade or division commanders of this affair mention, as a matter of course, the building of breastworks. Gen. J. E. Smith, commanding Third Division, Fifteenth Corps, says (Off. rec. U. & C., Ser. No. 55, p. 643), "The First Brigade deployed the Fourth Minnesota Volunteer Infantry as skirmishers so as to cover the brigade front, while the remainder of the First and Third Brigades intrenched themselves as rapidly as possible, so that by daylight, when the Second Brigade had crossed, the whole command was perfectly secure behind a good line of works. Daylight revealed a second ridge about 500 yards in advance of the first line. I ordered the command forward, and they again intrenched themselves." This report is especially interesting, as it shows the methodical manner in which the troops of this western army were already using fieldworks. Crossing the river in the middle of the night they did not await the coming of day to bury themselves, but at once set to work, and then when daylight revealed that the intrenched position was not the one tactically best, the first line was abandoned in a matter of fact way and the more suitable line occupied and intrenched. By 1 p. m. on the 24th this intrenched line had accomplished its mission, the entire command of Sherman was safely across and the advance was resumed.

Describing the action of the troops of his command during the remainder of the 24th, General Sherman says (*id.*, p. 573), "I had inferred that Missionary Ridge was a continuous hill, but we found ourselves on two high points, with a deep depression . . . The ground we had gained, however, was so important that I could leave nothing to chance, and ordered it to be fortified during the night. . . . Thus we passed the night, heavy details being kept busy at work on the intrenchments on the hill. . . . The farther point of the hill was held by the enemy with a breastwork of logs and fresh earth." From these reports we see that the troops under General Sherman intrenched three different lines in twenty-four hours, each position a little more advanced than the one preceding. On the 25th the corps of General Howard, ordered to assist General Sherman on the left, covered themselves with breastworks even though held back in reserve.

Passing now to the Confederate side we find two important lessons in the use of fieldworks. General Cleburne, assigned the task of covering the Confederate right along Missionary Ridge,

constructed one strong line of intrenchments on the heights near Tunnel Hill, the works consisting of log breastworks reinforced with earth dug from the trench inside and thrown in front of a log revetment. These works were, for the most part, constructed during the day and night of November 24, and were held by one division during all the next day. On this day, the 25th, six divisions under Sherman attacked the troops of Cleburne behind their intrenchments. So well did the latter serve their purpose that no troops, except in one instance, made a lodgment on the hill held by the Confederate intrenched line. The exception was in the case of the brigade under General Corse; this exception, however, but adds still further testimony to the power of hasty intrenchments. Smith, of Cleburne's division, early on the 25th threw up some slight advanced works from which he was later directed to retire in order to straighten the Confederate lines. These advanced works were seized by Corse's brigade in the first advance on the 25th; by a little work he reversed them, and though but about 100 yards from the Confederate main lines, his force maintained themselves in the captured trenches all day, repulsing several counterattacks.

The use made of fortified lines by the center of Bragg's army on the 25th is in sharp contrast to the successful use of them on the Confederate right. During the days before the arrival of General Grant, the Confederate Army had spent much labor in constructing intrenchments around Chattanooga. These works, along the center of Bragg's position directly east of Chattanooga, really consisted of three separate and distinct lines—one in the vicinity of Orchard Knob, one at the immediate foot of Missionary Ridge, and one on the summit. The first was merely an advanced line for strong pickets, and was captured on November 23. The last two constituted the main line; both were occupied in more or less force on the afternoon of November 25, and the Confederate forces holding them were approximately equal to the assaulting Union troops. In reviewing the operations of the contesting armies on that afternoon, we are struck not so much with the accomplishments of the field fortifications as with their failure. A knowledge of the cause of the failure is very important to the military student, and fortunately it is not difficult to discover. To our mind, it consists of the faulty tactical dispositions of the Confederate forces. The two lines of trenches, as occupied, not only were of no advantage, but in reality they were a detriment. Sufficient forces to resist the

Federal assault were not put into the first line. These troops, retiring directly to the rear and closely followed by the impetuous rushes of Thomas's men, masked the fire of their comrades in rear, causing the latter to fire high over their heads if they would avoid shooting their own troops. As General Grant says in his Memoirs, Vol. II, page 80, "In fact on that occasion the Union soldier nearest the enemy was in the safest position." Furthermore, owing to the location of the trenches on the summit, and the broken and diversified face of the hill, there were many ravines containing dead spaces almost up to the line itself. Thus no advantage was obtained from the second line, since the Federals followed the retreating first line so closely that the troops in the second line were unable to stop the Union advance with fire action, and this line, already thrown into confusion by the first line retiring through it, was an easy prey to the Army of the Cumberland.

The study of this battle discovers two important lessons. The first lesson is the defect inherent in a double intrenched line, by which troops retiring from the advanced line through the second line masks the fire of the second line and tend to communicate panic to it. The second is the difficulty of placing an intrenched line along a steep slope without leaving dead spaces. The second line of Bragg's army had been located by his engineer officer, and yet military critics, who have examined the still existing lines, state that in many instances the line, as strengthened, was not upon the military crest but behind it.

With the example of Sherman's Fifteenth Corps at Chattanooga fresh in their minds, and with Sherman's own personal experiences in the use of fieldworks at Vicksburg to draw upon, it is not surprising that we find the forces under his command during the Atlanta campaign making a more extended use of fieldworks than any which had yet been met with in the history of warfare. Writing of the extent to which fieldworks were used in this campaign, General Sherman in his Memoirs, Vol. II, page 56, states, "During this campaign hundreds, if not thousands, of miles of similar intrenchments were built by both armies, and as a rule whichever party attacked one of them got the worst of it." With the work of Johnston's army we will not deal, since they were primarily on the defensive and their works, in most cases, were more elaborate and stronger than ordinary field fortifications. The reason for this latter is obvious. Atlanta was the prize for which Sherman was striving, and to supply his numerous armies it was necessary

that his advance should follow closely the Chattanooga & Atlanta Railroad, and that he should control it as he moved forward. Johnston knew this, and in his defensive retreat made most excellent use of the knowledge. His decisions to retire were usually arrived at deliberately, and before his army withdrew engineer officers selected, laid out defenses and partially fortified the next position in rear. Thus it was that his retreat was from one prepared position to another, and an inspection* of the Confederate works along the route reveals the fact that these were always much more massive and elaborate than those of the Union troops. Another interesting feature brought out by careful examination of the works reveals also the fact that while the Confederates were prone to select the highest ridge in the locality, thus forcing the Federals to be content with the next best one, still the latter's trenches were all more skillfully placed as regards the accidents of the terrain. This bears out the statement of writers on the subject that trenches built under fire and by the troops that are to occupy them are generally superior from tactical standpoints to those laid out at some previous time.

The following statement of General Sherman in his Memoirs, Vol. II, page 57, well illustrates the use he made of fieldworks and furnishes us with a good vantage ground from which to approach a more exhaustive study of the Union lines at each successive position. General Sherman says: "... Aiming always to keep our fortified lines in absolute contact with the enemy, while with the surplus force we felt forward from one flank to the other for his lines of communication and retreat."

Sherman moved on May 5 against Johnston. Finding on the 7th the latter's position too strong to be attacked in front Sherman decided to turn it, and for that purpose he detached McPherson with the Army of the Tennessee via Snake Creek Gap. The latter had orders to seize Resaca or take position across the railroad in rear of Johnston. McPherson, deeming his force too small, took up a position covering the mouth of the gap, and fortified it. Sherman at once started the remainder of his army to reinforce McPherson, leaving only Howard with the Fourth Corps to cover his line of communications. The division commanders of the Fourth

*For the personal observations and inspections mentioned we are indebted to members of the Staff Class of 1906, who made the ride from Chattanooga to Atlanta.

Corps report (Vol. XXXVII of the Rebellion Records) that they at once strengthened their positions. These works were sufficient to enable the Fourth Corps to withstand the attack on them on the 12th. The flanking move was completed on the 13th, but the threat of it had already caused Johnston to abandon his strong works at Dalton and to retire to previously prepared positions around Resaca. Sherman advanced during the 14th and closed around the town. Of the actions on the 15th he says (Memoirs, Vol. II, page 35): "Without attempting to assault the fortified works, we pressed at all points, and the sound of cannon and musketry rose all day to the dignity of a battle." According to the reports of all three army commanders, despite the hard fighting of this day, much of the night was spent in intrenching the entire line from the Oostenaula River around to the Connasauga. During the night Johnston retired across the Oostenaula and burned the bridges.

Following the retirement of the Confederates from Resaca there is nothing of interest pertaining to our subject till the engagement near New Hope Church. Johnston had, however, prepared three strong positions; namely, Cassville, Etowah River line and Allatoona Pass. The first he abandoned owing to some misunderstanding with Polk and Hood. The latter two positions Sherman avoided by one of his favorite flank moves, though in this case, instead of leaving part of his force to intrench and cover the railroad, he took twenty days' supplies in his trains and cut loose from his lines, aiming to turn the Etowah River and Allatoona Pass positions and hit the railroad again south of Allatoona. Johnston heard of this plan in time to place himself across Sherman's front near New Hope Church and to construct formidable works. The advance of Thomas's army hit these fortifications on the afternoon of the 25th of May, but was unable to carry them.

From this time till June 1 the Federal troops were continually intrenching, gradually extending the left of their line toward the railroad near Acworth. In fact, it might even be said that from the 25th of May till the evacuation of Atlanta the entire movement of the Union Army was from one intrenched position to another. Careful measurements of maps, checked with reports of division, brigade and regimental commanders, indicate that in the vicinity of New Hope Church Sherman's "dirt diggers" in these seven days constructed from 25 to 30 miles of breastworks. No direct assault on the Confederate works was made, General Sherman, as he says, "having learned the folly of direct assaults against

such strongly fortified positions." In another part of his Memoirs, Vol. II, page 45, he says of the operations at this time: "All this time a continual battle was in progress by strong skirmish lines, taking advantage of every species of cover, and both parties fortifying each night by rifle trenches with head logs, many of which grew to be as formidable as first-class works of defense."

Johnston, finding that Sherman's continual extension "by the left flank" was bringing him to the railroad and nearer Atlanta than he was himself, evacuated his lines on the 4th of June and retired to his almost impregnable position in and around Kenesaw Mountain. This extension to the left was made possible by strengthening the parapets along the center and right, allowing this part to be held by fewer men.

The lines to which the Confederates retired on the 4th of June embraced the front slope of Kenesaw on the right and Lost Mountain on the left, with an advanced position or salient on Pine Mountain near the center. In the actions in this vicinity Johnston's army occupied virtually four different lines of trenches, all prepared beforehand; first, the position just mentioned, 11 miles in extent; second, a shorter line, Pine Mountain abandoned, directly connecting Lost Mountain and the first position on Kenesaw, 10 miles in extent; the third a line behind Mud Creek on his left, Lost Mountain being abandoned and still holding the front slope of Kenesaw, 6 miles in extent; fourth, the position along the summit of Kenesaw then west and south around Marietta, with left flank pointing directly south, 9 miles in extent. The strength of these works was such that General Sherman was unwilling to make a direct assault and resorted to his favorite maneuver, moving this time by his right flank. He made his works directly covering his advanced base at Big Shanty so strong that few men per yard could hold them, and extended his right, reaching for the railroad in rear of Johnston. The fieldworks here employed assume almost the dignity of siegeworks, especially when we consider the number of miles constructed—about 75. However, there is this important difference: in no case have we been able to find by search through the Rebellion Records where regular saps and approaches were resorted to. Lines were advanced and intrenched under fire, working parties being protected by very strong lines of skirmishers who kept down hostile fire; positions gained in an assault were held and intrenched under similar protection, or lines advanced at night to a position from which a sally could be made and intrenched under

cover of darkness. In fact, so onerous did this night work become that Sherman organized a pioneer corps for this work out of freed slaves, giving them rations and $10 per month.

A good example of intrenching under fire is described by General Howard in Off. rec. U. & C., Ser. No. 72, page 197, and more fully elaborated in Battles and Leaders, Vol. IV, page 309. He says: "Baird's division, in a comparatively open field, put forth a heavy skirmish line, which continued such a rapid fire of rifles as to keep down a corresponding hostile line behind its well constructed trenches, while the picks and shovels behind the skirmishers fairly flew, till a good set of works was made 400 yards distant from the enemy's and parallel to it."

The continual extension of the right having failed to induce Johnston to retire, Sherman reluctantly ordered an assault along his entire 10 miles of works on the 27th. The attack failed. General Sherman (Memoirs, Vol. II, p. 61): "satisfied of the bloody cost of attacking intrenched lines," decided on moving the whole army to Fulton, 10 miles south on the railroad. Before this move took place Johnston fell back into a previously prepared bridge-head covering the crossing of the Chattahoochee, leaving a small rear guard in position intrenched across the line of march near Smyrna. Thomas ran against this position on the night of the 3rd of July, and such was the habit of his troops to intrench that during this night they constructed about 6 miles of intrenchments. During the night of the 4th-5th of July all of Johnston's troops were withdrawn inside the *tête-de-pont*. General Sherman says this was the strongest piece of field fortification he had ever seen. He had had his lesson at Kenesaw attacking such works, and therefore at once resumed his former tactics intrenching a part of his force in close contact with the Confederate works and sending turning columns to cross the Chattahoochee both above and below Johnston's position. Both columns immediately on crossing the stream threw up extensive intrenchments as bridge-heads. The threats of these flanking columns were sufficient, and Johnston at once withdrew into the fortifications in the vicinity of Atlanta. These consisted of two distinct lines, the first one running from the Georgia Railroad, midway between Atlanta and Decatur, northwest to the Western and Atlanta Railroad, near the Chattahoochee; the second was a line completely encircling the city, constructed on a very elaborate scale nearly one year previously. Hood, who relieved Johnston about this time, further extended his

lines about 10 miles to the south along the railroad to Eastpoint, trying to meet and stop Sherman's advance by the right flank.

In all the operations of the Union Army in front of Atlanta we find it putting into operation the lessons so well learned in the previous battles of this campaign. At no time were serious direct assaults attempted against the Confederate breastworks. However, there were several instances of Confederate assaults upon the Federal lines, but the works of the latter, though much inferior in profile to those of Hood's forces, were invariably able successfully to withstand the assaulting columns. Despite the fact that the defenses of Atlanta approached in magnitude semipermanent fortifications, still the Union troops did not adopt any of the measures of regular siege operations. Captain Poe, Corps of Engineers, U. S. A., reporting on the Atlanta campaign, says (Off. rec. U. & C., Ser. 72, p. 134), "The general (commanding) . . . always told me that he did not wish anything of the kind done (referring to siege operations), that he intended to gain possession of Atlanta by operating upon the enemy's lines of communications, . . . to bring on a general engagement . . . or force the enemy to evacuate the city . . ."

As illustrating the extent to which the use of hasty intrenchments had grown at this time, another extract from the report of Captain Poe, quoted above, is illuminating. He says: "The troops intrenched their position every night. This was made a rule from the time the campaign commenced, and was continued until the close of the war whenever the proximity of the enemy made it prudent. I may add, also, that during all the operations of this great army, extending over a year of time and thousands of miles of territory, it was never surprised."

During the progress of this campaign the habit of meeting the enemy from behind breastworks, when they made an offensive return, had become so universal that General Sherman, in his report on the Confederate attack of July 20th, mentions as an unusual fact that "Hooker's corps was actually uncovered, and had to fight on open ground, losing about 1,500 men."

The Confederate employment of fieldworks on the defense was so extensive that one historian has remarked that no adequate conception of this remarkable campaign can be obtained unless the formidable nature of these artificial defenses is remembered. As for their use on the offensive, General Sherman remarks, "The

skill and rapidity with which our men constructed these is wonderful, and is something new in the art of war."

Passing from the considerations of the operations of Sherman's army to the eastern theater, we find the forces under Generals Grant and Lee in the last great campaign of the war showing a full appreciation of the offensive and defensive value of field works.

Early on the morning of the 4th of May, 1864, the Army of the Potomac, consisting of the Second, Fifth, Sixth and Cavalry Corps, began its crossing of the Rapidan, and by nightfall the entire army was on the southern side. The following morning the Fifth Corps, which was in advance, encountered Ewell's corps marching on the Wilderness via the Orange Turnpike. Two divisions under Hill were marching on the same objective via the Orange Plank Road, about 3 miles to the south of and parallel to Ewell's line of march. Crawford's division of the Fifth Corps, moving to the south, gained contact with Hill's column. Warren was ordered to halt and prepare to meet and attack the enemy, and about noon the corps having deployed, moved forward, the Orange Turnpike being the axis of the movement. The country was very thick, and good troop-leading was difficult or impossible. The advance of the Fifth Corps was checked. The Second and Sixth Corps were deployed, respectively, on the left and right of the Fifth, and Burnside's corps was ordered to hasten its march across the Rapidan. Notwithstanding the reinforcements, the National offensive was unable to gain ground. "Slight works were at once thrown up, and several partial attacks of the enemy repulsed." (Report of Ewell, Off. rec. U. & C., Ser. No. 67, p. 1070.) "All of Hancock's corps were directed to throw up breastworks of logs and earth, the intrenched line beginning at Getty's left and extending to Barlow's left, where it was refused to cover the flank. The second line of the Second Corps also threw up earthworks, and a third intrenched line was formed behind Birney and Mott nearest the Plank Road" (General A. S. Webb, B. & L., Vol. IV, p. 157). Much of this work was done under fire.

It may be truthfully stated that from this time until April 9 of the following year, 11 months and 5 days, the Army of the Potomac and the Army of Northern Virginia never halted during the operations without covering themselves with field fortifications. The lesson, after three years of instruction, had been finally learned.

In the expectation that they would be relieved by troops of Longstreet during the night, no effort was made by Hill's troops to strengthen their position. On the other hand, Ewell's works

were "greatly strengthened." The National troops resumed the offensive on the morning of the 6th, but without success. During the afternoon troops of Longstreet's corps, which arrived during the morning, enveloped the National left, but were unable to make any progress against the second line, which had been prepared for defense, and were driven from the advanced lines by the National reserves. In the evening the Confederates gained a slight advantage on the Federal right. During the night of May 6-7 the Federal cavalry was withdrawn from Todds Tavern and the Brock Road and likewise intrenched.

On the 7th the opposing forces remained in their works, the National right having been withdrawn, and during the evening Grant set the army in march in the direction of Spottsylvania Court House, the movement beginning from the right flank. Stuart's cavalry promptly reported this movement, and the Confederate Army moved by the right flank, but with the right in front. Both armies were moving in parallel columns on a common objective. After an all-night march Longstreet's corps arrived at Spottsylvania at about 8 a. m., and found Stuart opposing the advance of the Fifth Corps, which was in the lead of the National column and which, being unable to gain ground, immediately commenced intrenching (Law, B. & L., Vol. IV, p. 128). Both sides hurried forward troops May 9, and the day was largely spent in intrenching the lines, and in reconnaissance. The Fifth Corps assaulted the Confederate works late in the afternoon of the 10th, but the attack was unsupported and failed. An attack by Upton's brigade and other troops of the Sixth Corps penetrated the Confederate lines but, being likewise unsupported, had to relinquish the advantage gained and withdrew under the cover of darkness. The 11th was spent by the National forces in reconnaissance with a view to ascertaining any weak points in the Confederate lines. It was found that in the vicinity of the McCool house there was a considerable salient. It appears that in the hasty extension of the Confederate lines on the 8th "Ewell, to keep on high ground, had changed direction and gone a mile north" (Alexander). As the troops were disposed on the 11th, Ewell was in the center of the Confederate line. His forward position caused the salient because of the necessity of covering Spottsylvania Court House with respect to Burnside's corps on the National left. "Ewell ought to have disregarded the high ground and continued the intrenchment eastward along the

trace adopted . . . It must not be forgotten that the direction of a line is often of far more consequence than its elevation." (Steele.)

Hancock was ordered to attack the salient at 4 a. m. on the morning of May 12, supported by Warren and Wright. The attack succeeded in capturing the salient, but was spent when it arrived before an interior line connecting the Confederate works to the right and left of the same. Warren and Wright were unable to accomplish anything, and it may be stated that the Confederate lines held. May 19 the Army of the Potomac again moved by the left flank. May 24 found the armies facing each other on the line of the North Anna. Both intrenched. Grant decided the Confederate position was too strong to assault, and on the 26th continued his movement by the left flank. On the 30th the National troops again encountered Lee's forces in a strongly intrenched position on the south bank of the Totopotomoy. Grant decided again not to assault.

On June 1 the Sixth and Eighteenth Corps made a determined attack on the hastily prepared position at Cold Harbor, but were repulsed, and themselves intrenched. The army having been concentrated, a general assault was made on the Confederate position, which had been considerably strengthened, on the morning of June 3. This was repulsed with a loss of 6,000 killed and wounded in less than an hour. The armies confronted each other in their works until the 12th, when the National Army again marched by the left flank, moving in rear of works prepared by the Fifth Corps, which had been withdrawn from the main lines on the 7th for the purpose of constructing them.

Early in the morning of June 15, the Eighteenth Corps began to arrive before Petersburg. In addition to his Eighteenth Corps, which had about 10,000 effectives at that time, Smith had also Kautz's cavalry, about 2,400, and a part of Hink's negro division, 3,700 rifles. The Confederate garrison consisted of the Twenty-sixth, Thirty-fourth and Forty-sixth Virginia, the Sixty-fourth Georgia, the Twenty-third South Carolina, and some extemporized troops, in all about 2,200. "These troops occupied the Petersburg line on the left from battery No. 1 to what was called Butterworth's Bridge . . . and had to be so stationed as to allow but one man for every 4½ yards. From that bridge to the Appomattox—a distance of fully 4½ miles—the line was defenseless" (Gen. G. T. Beauregard, B. & L., Vol. IV, p. 540). The following description

of the lines at that time is given by Brig. Gen. R. E. Colston, C. S. A. (B. & L., Vol. IV, p. 535), "At that time (June 9, 1864) the lines covering Petersburg on the south side of the Appomattox formed a semicircle of about 8 miles development, resting upon the river at each extremity. With the exception of a few lunettes and redoubts at the most commanding positions, they were barely marked out, and a horseman could ride over them without the least difficulty almost anywhere, as I myself had done day after day for weeks just before the fight. They differed *in toto* from the shortened and formidable works constructed later by General Lee's army."

Smith's command was not supported by other National troops on the 15th, and because of poor reconnaissance and worse troop-leading the works were not assaulted until about 7 o'clock in the evening. By 9 o'clock the Federal troops had succeeded in making a lodgment, and gained control of about 1½ miles of the Confederate works. The attack, however, spent itself in the capture of the works, and the advantage gained was not followed up. During the evening the garrison was reinforced. Two days later the Eighteenth Corps was relieved from duty with the Army of the Potomac and returned to Butler in the vicinity of Bermuda Hundred. Beauregard states, concerning the Confederate reinforcements, "The digging of trenches was begun by the men as soon as they reached their new position. Axes, as well as spades; bayonets and knives, as well as axes—in fact, every utensil that could be found was used." The city speedily became a fortress, and siege operations were resorted to by the Federal Army.

The events of the 15th are of more than passing notice, as they present a notable example of the use of supporting points. The Confederate lines were without curtains, and consisted of works located at intervals of from 200 to 400 yards. These were appropriate for the ranges of the small arms of the period. The defensive power of the small garrison was increased to the utmost by the discontinuous line, and the most modern principles of field fortifications differ only in prescribing greater intervals between works, due to increased ranges and rapidity of fire, and the dispositions of the artillery.

Here the armies confronted each other for several months, during which time parts of the National forces were moved to the north and west with a view to extending and weakening the Confederate lines. In March, 1865, these movements had caused an

extension of the Confederate lines to a length of more than 35 miles. The returns of February 28, show:

	Effectives.
Army of Northern Virginia	45,057
James River defenses (estimated)	5,000
Richmond (estimated)	3,000
Total	53,057

or about 1.1 men per yard, against which the National forces were able to make no headway, and were required to march again by the left flank with a view to cutting the Confederate lines of supply. The Confederate works at Five Forks were turned, not carried, and for the first time in eleven months the National troops succeeded in forcibly ejecting their opponents from a position by direct operations upon the point occupied.

During the night of April 2-3, 1865, the Confederate forces evacuated their works in the vicinity of Petersburg and the James, and retired to the west. The pursuit was promptly taken up by the National Army. To the end the Confederates resorted whenever possible to defensive works. Humphreys states (p. 379, Virginia Campaign of '64-'65), "(After crossing Flat Creek on April 6) a sharp running fight commenced at once with Gordon's corps . . . during which several partially intrenched positions were carried, 'later,' at 11 o'clock (on the 9th) the Second Corps had come up with Longstreet's command intrenched in the vicinity of Appomattox Court House." The Second and Sixth Corps were at once deployed for attack, and were about to advance, when the movement was ordered suspended by Meade, a truce having been entered into which resulted in the surrender of the Confederate forces.

With the surrender of Lee came the end of the Civil War. The history of these four strenuous years shows us raw and untrained levies going into the great melting pot of war and, at the end, coming out of the crucible disciplined, trained and war-hardened veterans, forming one of the finest armies known to modern times. At the beginning of the struggle, our National forces were but crude when compared with the troops of almost any European power; at the end it is doubtful if the combined armies of Europe could have prevailed against the united forces of the North and South. Our experience and teachings in all branches had been received in the great, grim school of war and our lessons finally had been well

learned. In no direction was the increase in knowledge more marked than in the one under consideration.

An incident worthy of note in this connection, is that the two most famous Federal generals—Grant and Sherman—were both present, in commanding rôles, in one of the first great battles of the war. In that battle (Shiloh) the disaster, which was the logical result of Union neglect to employ field fortifications, was narrowly averted, solely because the arrival of Union reenforcements completely changed the problem. In the last year of the war these same generals—one in supreme, the other with an independent command—employed hasty intrenchments to such an extent as to make their use the marvel of the military world.

For us at the present day, this dearly acquired knowledge, gained only after years of sanguinary conflict, has a real application, more especially since our greatly improved firearms render the use of hastily constructed fieldworks of even greater importance than ever before. To our mind the lesson is clear that even the most elementary instruction which an officer now receives should include a course in the theoretical and practical application of the use of fieldworks to tactical problems. The course of Field Fortification at the Army Service Schools forms the basis for this instruction and, as graduates of this school return to duty with troops, we feel that they could do no greater good to the personnel of their regiments than to start their brother officers in an elementary course in field fortification problems, actually worked out on the terrain in the vicinity of their posts, and we unhesitatingly recommend that a practical course in field fortification, based on tactical considerations such as is conducted at the Service Schools, should be included in the Garrison School for Officers.

The present military policy of the United States is such that in our next war the use of field fortifications, because of the extemporized character of the troops with which we are certain to enter the war, will be the rule rather than the exception.

The importance of field fortifications has been still more convincingly demonstrated by recent wars in South Africa and Manchuria. In the present European war, field fortifications have been developed to a point beyond the dream of military men of the last generation; to such extent that the hostile operations on both sides partake virtually of the nature of siege works on the most stupendous scale.

This increase in the importance of field fortifications has been

accompanied by a corresponding increase in the number of intrenching tools provided for combatant units. Instruction in the use of fieldworks and in what we may safely hope to accomplish with the implements furnished has not, however, increased in the same ratio. Our army is now plentifully supplied with tools, both park and portable; the teachings of our last great war, now even more important than fifty years ago, are such that the knowledge when and how to use these tools offensively and defensively should be universal throughout the service.

The Use of Outlets for Reducing Flood Heights.

BY

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The controversy as to the efficacy of outlets, or in other words the diversion of a portion of a flooded stream to secondary channels, for the purpose of reducing flood heights, began in Europe several centuries ago and was transferred to this country during the early days of levee building in Louisiana over a hundred years ago.

In Europe, outlets for flood relief as applied to alluvial streams flowing in beds of their own formation, were long since rejected through the actual experience of eminent hydraulic engineers.

Gen. Q. A. Gillmore, then President of the Mississippi River Commission, before the Senate Commerce Committee in 1882, stated:

It is perhaps well at this time, when certain engineering heresies are struggling into unusual and mischievous prominence, that we should again verify our bearings, and re-examine, and if admissible, reform these physical laws upon which the plan of improvement of the Mississippi River recommended by the Commission is avowedly based, because upon the soundness and correct interpretation of those laws that plan unquestionably rests for success. The Commission, in its first report, restated a well-known law as follows:

If the normal volume of water in a silt-bearing stream flowing in a bed of its formation be permanently increased, there will result increase of velocity and consequently of the erosive and silt-bearing power; an increase of depth, if the banks are held, and an ultimate lowering of the surface slope; and conversely, if the normal flow be decreased in volume, there will ensue a decrease in velocity, silt-transporting power and mean sectional area and an ultimate raising of the surface slope. . . .

This law applied to the Mississippi River is exhaustive and all-sufficient. It solves all questions and rejects all fallacies. . . . It condemns outlets because they abstract water from the river, thereby lessening the volume, velocity, flow and scouring power, and cause deposits below the outlets in proportion to the volume abstracted.

*A paper read before the Mississippi River Commission at its 126th session.

The last great battle of outlets on the Mississippi River was fought about a quarter of a century ago, in which the outlet advocates were defeated. Since that time the subject has been very little discussed until the great floods of 1912 and 1913 brought the subject up again under the guise of spillways or waste weirs for the reduction of flood heights.

No new facts regarding the regimen of the river or the problem of flood control have been developed; not even the greatest of all recorded floods, that of 1912, reached a stage at Carrollton exceeding the provisional levee grade established twenty years ago; no difficulties have been encountered in levee construction suggesting that flood control by means of levees is impracticable; no new or unlooked for conditions have developed in the stream itself. So take it all in all, it is difficult to find a reason for any change at this time, in the general plans which were laid down in the early days of the Mississippi River Commission, and which have been satisfactorily carried out as rapidly as available funds would permit.

The good people of New Orleans have, however, recently become alarmed, due perhaps largely to the teachings of reservoir enthusiasts who predict disaster unless flood heights are materially reduced rather than being controlled and guided safely to the sea. This situation might readily develop to such extent as to become a real menace to the welfare of the city, whether their fears were groundless or not. Rival Gulf ports would doubtless make the most of all drawbacks that might be chargeable to the conditions at New Orleans, and this also aggravates the situation.

Whatever may be the cause for the alarm, and whether based on facts or not, its existence justifies a review of the situation for the purpose of reaffirming the governing principles which have been held so long if found to be correct, or, if deficient, to develop such remedies as may fully meet all requirements.

Such review has been undertaken by the writer for his own information, and as a large mass of expert testimony of men of large experience in river hydraulics has been accumulated, it is submitted to the Commission for the information of its members.

The writer had accepted the elaborate investigations of the Mississippi River Commission covering a large number of crevasses under varying conditions, both open and closed, as fully establishing the principle that no permanent lowering of the flood plane would result from the use of outlets, spillways, or waste weirs.

A large mass of testimony of eminent hydraulic engineers before Congressional Committees, seems also to point to the same con-

clusion. At the end of a prolonged Congressional hearing in 1882, Congressman McLane of Maryland exclaimed "That is the end of the outlet schemes," but the subject comes up again in a modified form which requires consideration.

Spillways or waste weirs would differ from crevasses only in the fact of having the outflow controlled so as to discharge a given amount and at the same time confine the water to the floodway prepared therefor. The effects on the main stream would necessarily be the same in either case.

No new facts have been developed which controvert the fundamental law of hydraulics, that the reduction by means of secondary channels of the volume of an alluvial stream flowing in a bed of its own formation will reduce the capacity of the main stream, which will ultimately result in an increased flood height at and near the point of diversion and throughout the channel below the same.

We have an illustration of this law at Red River Landing, where upwards of 300,000 second-feet are at flood times diverted from the Mississippi down the Atchafalaya to the Gulf of Mexico.

Discharge measurements have been taken at this point during seventeen flood years dating as far back as 1882.

The following table gives the stages for given volumes of discharge as measured in the several flood years:

Red River Landing.

Year.	Discharge 1,000,000 s. f.	Discharge 1,100,000 s. f.	Discharge 1,200,000 s. f.	Discharge 1,300,000 s. f.
	Stage.	*Stage.*	*Stage.*	*Stage.*
1882	39.3	41.3	43.2	45.0
1885	39.5	41.2	42.5	46.8
1890	41.2	44.0	45.9	----------
1891	----------	44.5	----------	----------
1892	40.7	44.3	47.0	48.3
1893	43.3	45.0	47.3	----------
1897	----------	----------	47.3	50.5
1898	----------	44.2	----------	----------
1903	----------	49.5	50.0	----------
1904	43.5	44.6	46.5	----------
1906	43.8	44.4	----------	----------
1907	----------	----------	46.9	46.9
1908	----------	45.0	46.8	----------
1909	----------	45.9	----------	----------
1911	41.8	42.5	----------	----------
1912	----------	----------	50.0	53.6
1913	48.4	50.4	50.2	50.6

This table shows that prior to 1890, volumes of 1,000,000 second-feet and over were passed at a much lower stage than has been the case in later years. In other words, the stage for a given volume of discharge has increased, indicating a material elevation of the flood plane at Red River Landing.

Similar data for Vicksburg and Arkansas City, which are free from continuous outlet influence, do not show any substantial progressive elevation of flood plane.

The following tables give the available results at the points named:

Vicksburg.

Year.	Discharge 1,300,000 s. f.	Discharge 1,400,000 s. f.	Discharge 1,500,000 s. f.	Discharge 1,600,000 s. f.	Discharge 1,700,000 s. f.
	Stage.	Stage.	Stage.	Stage.	Stage.
1890	47.7	-------	-------	-------	-------
1897	-------	-------	49.3	49.4	50.0
1898	-------	49.5	-------	-------	-------
1903	-------	-------	-------	51.4	-------
1904	-------	46.6	-------	-------	-------
1906	-------	46.7	47.0	-------	-------
1907	-------	-------	-------	49.5	49.6
1908	47.3	47.5	-------	-------	-------
1909	46.7	47.7	48.0	-------	-------
1912	-------	51.0	51.3	-------	-------
1913	-------	-------	48.6	-------	52.1

Arkansas City.

Year.	Discharge 1,200,000 s. f.	Discharge 1,300,000 s. f.	Discharge 1,400,000 s. f.	Discharge 1,500,000 s. f.	Discharge 1,600,000 s. f.
	Stage.	Stage.	Stage.	Stage.	Stage.
1887	-------	-------	46.5	-------	-------
1890	48.7	48.0	48.5	-------	-------
1891	47.0	47.0	48.0	-------	-------
1892	47.2	46.0	47.0	48.0	48.0
1893	46.2	47.7	49.0	49.7	50.0
1897	-------	-------	-------	-------	51.0
1898	-------	-------	-------	51.0	-------
1903	49.0	47.2	48.5	51.2	51.8
1904	45.8	47.1	49.0	-------	-------
1906	-------	-------	49.4	50.0	-------
1907	45.0	48.1	48.5	50.8	51.8
1908	-------	47.0	-------	-------	-------
1909	-------	48.1	49.2	49.9	-------
1911	47.0	47.0	48.0	49.2	49.9
1912	-------	-------	-------	50.0	51.1
1913	-------	46.9	48.6	49.9	52.3

The absence of figures in these tabulated columns indicates that discharge observations are lacking. The figures given are deduced from the standard discharge curves of the several stations as developed from the observations of each successive year. The results are doubtless modified by the effect of crevasses in the vicinity of the discharge stations, which largely accounts for the fluctuations that appear.

In the Mississippi River below Bayou Sara, the question of interference with navigation does not enter in, as the depth, except perhaps at the Passes, will always be ample.

But the depleted section of the channel below the point of diversion can only be restored by a return to the volume existing before the diversion to the secondary channel occurred. No scour of a lesser volume can completely restore the channel to its original capacity, and the ultimate result will be that the permanent reduction of flood height hoped for will not be realized.

This seems to be the conclusion reached by a large majority of those whose testimony is quoted, who by experience and training were qualified to analyze the experiments and investigations that have been made relative to crevasses.

Gen. C. B. Comstock wrote the minority report of the Commission in 1893, which states—

> We do not wish to be understood as condemning the use in the levees of long waste weirs to take off the top of the flood if it shall be found that at certain places in the lower part of the river the further increase in flood flow which will come from raising the levees at points further up the river can be controlled in whole or in part by such waste weirs more economically than by higher levees.

In answer to a question by the Senate Commerce Committee as to "How deep an outlet could you have there without shoaling the river below?" General Comstock said: "It is possible you might take out 100,000 cubic feet per second. You might take 100,000 cubic feet per second without injury to South Pass. I do not think you would gain enough by that experiment to balance the danger you run."

The spillway advocates rest their claim of flood relief solely on the immediate lowering of flood height which generally takes place in the case of crevasses.

This has been observed in nearly every crevasse that has occurred, but the amount of flood depression due thereto is largely

over-estimated, and is much smaller than the diverted volume would seem to demand. This fact alone indicates that the channel capacity of the river has been materially reduced by the outflow through the spillway.

A crevasse occurred at Panther Forest, 150 miles above Vicksburg, on April 12, 1912. The estimated discharge of this crevasse was 101,026 second-feet.

Salem Crevasse, 35 miles above Vicksburg, occurred on the same day, and carried 301,806 second-feet. Both of these crevasses were in the right bank levee line, and discharged into the Tensas Basin opposite Vicksburg.

A discharge measurement of the Mississippi River was made at Vicksburg before the breaks occurred, and showed a volume of 1,780,407 at a stage of 51.5; two days after these crevasses occurred the stage had fallen but 0.5 foot, but the discharge was 369,220 second-feet less.

The Nita Crevasse which occurred in 1890 reached a discharge of 402,556 second-feet, yet the flood stage at Carrollton, about 58 miles below, was lowered only 1.6 feet. The crevasse carried about one-third of the discharge of the entire river.

This may be regarded as a fair measure of the volume which must be diverted through spillways in order to secure even temporary relief relatively small in amount.

In order to appreciate what it means to safely conduct such an enormous volume across the country to tide water, it is well to compare it with streams which are somewhat familiar to us.

Red River, which is navigable for several hundred miles of its length, has a maximum flood discharge of 226,000 second-feet, a little over one-half the volume of the Nita Crevasse.

The Colorado River, which created such havoc in Imperial Valley, California, has a maximum discharge of 150,000 second-feet, or about one-third of the crevasse. The Niagara River which, with its cataracts and rapids, is one of the wonders of the world, has a normal discharge of about 200,000 second-feet, or one-half of the discharge of the Nita Crevasse.

In order to divert so large a volume through a spillway whose average depth would be limited to 10 feet or less, it would be necessary to provide a very wide opening.

The river end of the spillway, for 2 miles or more, would have a bottom slope of over 5 feet per mile and a surface slope at flood stages of about double that amount.

The high velocities corresponding to such great slopes would make it both difficult and expensive to protect the bed and controlling side embankments from destructive erosion. Limiting the spread of the outflow between embankments would accentuate the tendency toward a diversion of the whole stream, and in this particular the spillway would differ materially from a crevasse, whose waters are distributed over hundreds of square miles and are not limited to any well defined channel except in the immediate vicinity of the break.

On the other hand, as the spillway recedes from the river and approaches Gulf level, the diminished slope will cause vast deposits of sediment. Such deposits will also result from diminishing flow in the wide, shallow channel as the flood recedes, and each succeeding flood will find a depleted channel with sand bars and islands common to alluvial streams, the ultimate effect being the virtual closure of the spillway as an efficient outlet.

It is estimated that an outlet flowing 100,000 second-feet for sixty days will carry about 1,000,000 cubic yards of solid matter, which will be deposited en route or when it reaches the lake or Gulf waters. To care for this enormous quantity each year by artificial means would evidently be impracticable in case of a flow of 400,000 second-feet.

Cubitts Gap and The Jump have been practically closed through such natural processes.

The sites for spillways which have been most frequently considered are Bonnet Carre (926 L*), 31 miles above New Orleans, where the bend of the Mississippi River comes within 4½ miles of the west shore of Lake Pontchartrain, and Lake Borgne (980 L), about 7 miles below the lower limits of New Orleans, where English Turn is only 5¼ miles from the lake.

Lake Pontchartrain has an average depth of about 15 feet, and the distance from the spillway to the 18-foot curve of depth in the Gulf of Mexico would be about 80 miles.

Lake Borgne and Mississippi Sound average less than 10 feet in depth, and the distance from the spillway by this route to the 18-foot curve in the Gulf is about 50 miles.

Another site which has been considered is in the vicinity of Kenner (947 L), 10 miles above New Orleans, where the distance from the river to Lake Pontchartrain is a little less than 5 miles.

The effect of either of these spillways would be to raise the level of Lake Pontchartrain from 2 to 4 feet, depending on the volume of outflow, and its 100 miles of shore line would be submerged to

*Miles below Cairo, Ill., left bank.

such extent as to cause very material damage to the industries along its shores, including the city of New Orleans, a large area of which is but little, if any, above Gulf level.

The lakes named are so shallow that they provide comparatively little storage capacity for the enormous volume of sediment which would be deposited in the course of a few years.

A site for a spillway on the right bank has been suggested in the vicinity of Jefferson (951 R), about 6 miles above the northern limits of New Orleans.

The river here is about 50 miles from deep water in the Gulf of Mexico, mostly through swamps and lakes but little above tide water.

Guide levees would be required for a considerable part of this distance. Just what difficulties would be encountered in their construction has not been determined.

Another possible site lies below Jesuits Bend (999 R), some 26 miles below the lower limits of New Orleans. The distance from the river at this point to Barataria Bay is about 15 miles, and to deep water in the Gulf of Mexico, 27 miles.

The surface slope of the ground from the river to the swamp, a distance of about half a mile, is 5 feet.

The natural spillways or outlets have been largely confined to the right bank of the river, where we find the Atchafalaya, Plaquemine, and Lafourche, only the first of which is now effective.

Bayou Plaquemine was closed in 1865, and Bayou Lafourche in 1904, while the closure of the Atchafalaya has a large number of advocates.

To close these natural outlets and substitute artificial ones seems rather inconsistent, to say the least.

While spillways could be constructed on the left bank at less cost than on the right bank, owing to the shorter distance from the river to tide water, the right bank has the advantage of less possible damage due to backwater.

The influence of a crevasse or spillway in reducing flood heights extends but a short distance upstream. Hence if general relief is its purpose, its location should be as far up the river as practicable. If its chief purpose should be to prevent the submergence of the wharves at New Orleans, then the spillway should not be remote therefrom.

It is possible that the difficulty could be met more economically by raising the wharves.

The question of the necessity of reducing the flood heights as

presented at this time is unique. Nothing has occurred, even during the great floods of 1912-1913, which have scored the highest record up to date, to justify a fear that properly constructed levees will not prove effective in protecting the delta of Louisiana and the city of New Orleans, with its great property interests, from overflow. The suggestion of such a thing would be sharply resented.

But the wharves of New Orleans lie between the protecting levees, with a grade line so low that a confined flood without crevasses from Cairo down might, on rare occasions, slightly overtop their decks. This is apparently the immediate cause of a revival of the outlet theory under the name of spillway or waste weir.

Its advocacy at this time, when the Government is being urged to provide for the completion of a levee system, is unfortunate. It will tend to discredit the levee system and if there should be hesitation in providing for its completion, it would naturally be chargeable to these differences of opinion which suggest that the problem of flood control has not yet been solved.

The present relations between the decks of the wharves along some 6½ miles of the river front of New Orleans are shown in the following tabulation of elevations of the wharves and the highest known stage of water, which was that of 1912, both referred to the Cairo datum. These data were furnished by Mr. Frank M. Kerr, Chief State Engineer.

Name of wharf.	Elevation of deck.	Elevation of high water, 1912.
	Feet.	*Feet.*
Valance Street	45.0	40.5
Harmony Shed	41.4	40.3
Sixth Street	42.0	40.37
Soraparu Street	42.0	39.99
St. Andrew Street	42.0	----
Celeste Street	41.5	----
Wm. A. Kernaghan	41.75	39.98
Erato Street Shed	41.75	40.04
Branch M. King Shed	41.5	40.09*
Hugh McCloskey Shed	41.5	40.14†
Toulouse Street Shed	41.5	40.02
Picayune Street	41.75	----
Gov. Nichols	40.04	40.04
Mandeville Street Shed	41.5	39.58
Foot of Desire Street	41.5	39.21
Foot of Pauline Street	43.5	----

*Poydras. †Canal Street.

The margin between the decks of the wharves and the maximum high water vary from 0.0 at the Gov. Nichols dock to 4½ feet at the Valance Street dock, and averaging about 1¾ feet above the high water of 1912.

This margin is equivalent to an increase of over 4 feet in the extreme flood stage at Red River Landing, so the danger of seriously overtopping the wharves seems quite remote.

If all of the wharves had the 4½-foot margin shown by the Valance Street wharf there would be no danger at all of being flooded.

Conditions might arise where human lives were endangered and the safety of large property interests was threatened by an unusual and excessive flood, where the diversion of a portion of the flood to and over a comparatively undeveloped district would be justified. For instance, if the destruction of a city was threatened and could be prevented by diverting the water through a spillway, there should be no hesitation in doing so. Great conflagrations have been checked by the destruction of blocks of valuable property in its pathway.

The law of self preservation has in past years been indulged in by people on one side of the river cutting the levees on the opposite side. A better levee system, and the increased confidence in its efficiency, has practically eliminated danger from this source.

An examination of the accompanying hydrographs of the great crevasses that have occurred below the mouth of Red River during the pasty thirty years shows such small reduction in flood height as to offer little encouragement for the construction of costly spillways. The effect there shown is for the single flood season when the break occurred. It is practically certain that they would grow less and less effective during succeeding flood seasons.

The Davis Crevasse of 1884, 15 miles above New Orleans, discharging 140,156 second-feet, lowered the flood about 1 foot. The Morganza Crevasse at the same time was discharging 211,109 second-feet.

The Nita Crevasse, 58 miles above Carrollton, which occurred in March, 1890, and discharged 402,586 second-feet, lowered the stage at Carrollton 1.0 foot.

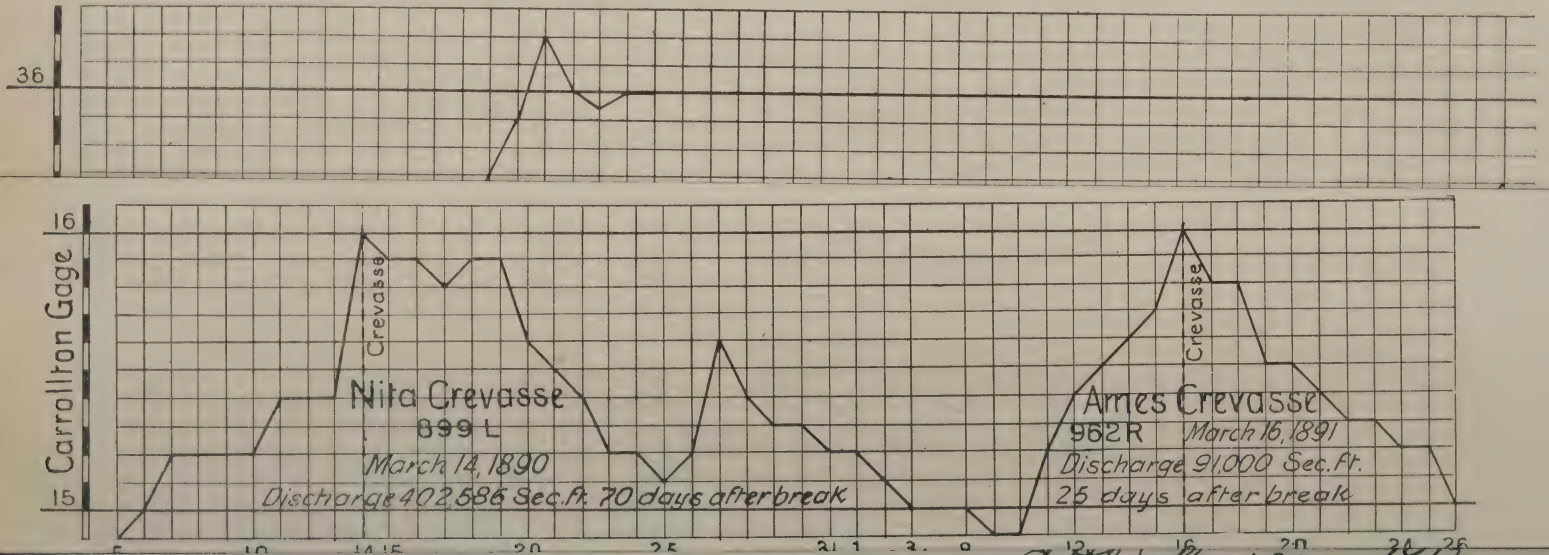
The Ames Crevasse, about opposite the Carrollton gage, which occurred in March, 1891, lowered the flood about 1 foot with a discharge of 91,000 second-feet.

The Belmont and Sarpee crevasses, which occurred June 12 and

HYDROGRAPHS OF CREVASSES

BELOW RED RIVER LANDING

SHOWING EFFECT ON FLOOD HEIGHTS

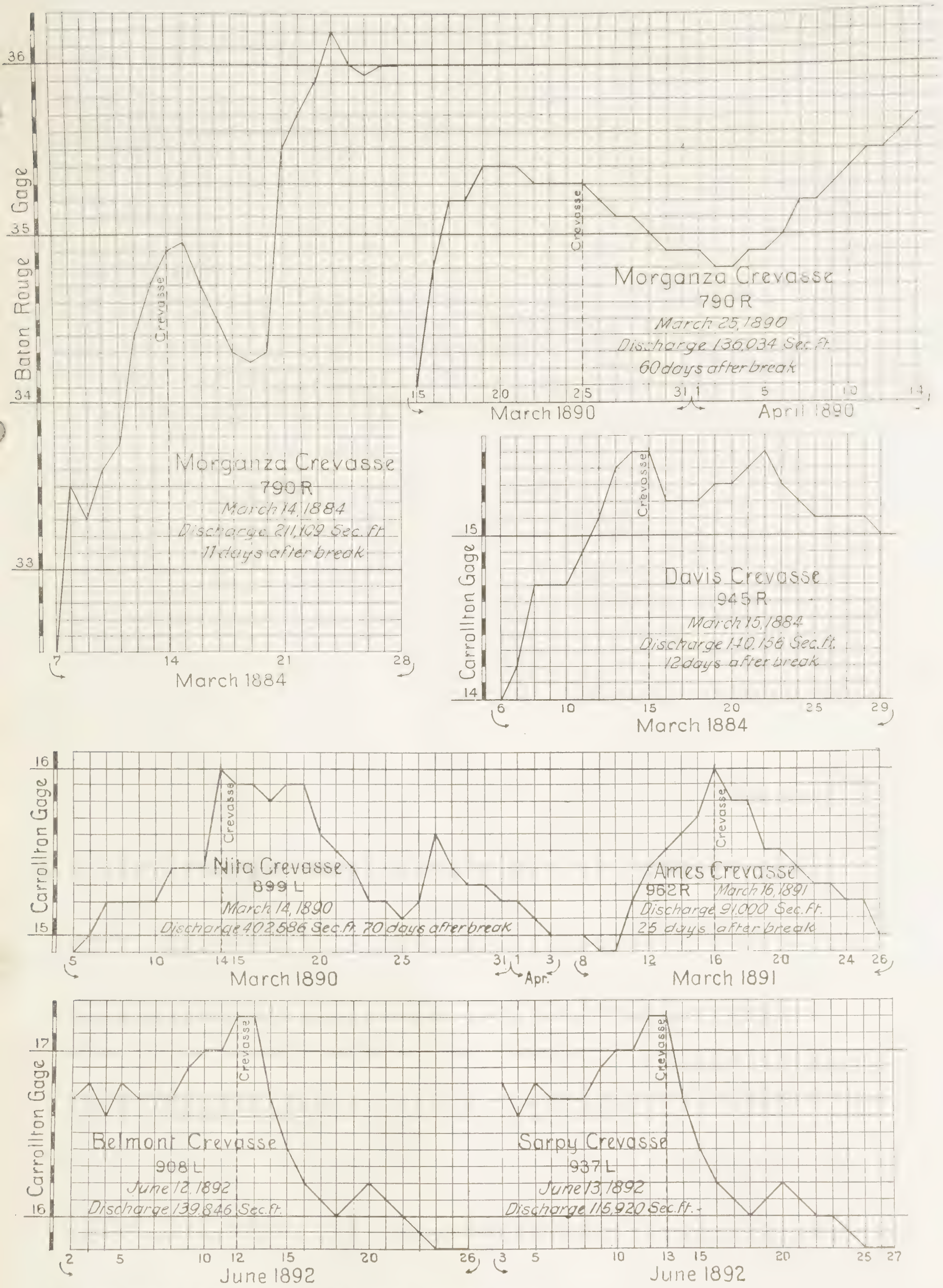


J. H. McReason 1913

HYDROGRAPHS OF CREVASSES

BELOW RED RIVER LANDING

SHOWING EFFECT ON FLOOD HEIGHTS



13, 1892, with a combined discharge of 255,766 second-feet, lowered the flood about 1.2 feet at the Carrollton gage, 50 and 20 miles below, respectively.

The Hymelia Crevasse of March, 1903, discharging 30,444 second-feet, lowered the stage 0.4 foot at Carrollton, 32 miles below. It flooded 1,320 square miles of land. And the crevasse of May, 1912, discharging 74,000 second-feet, lowered the stage at Carrollton 0.5 foot.

While the writer has the utmost confidence in the efficacy of levees to protect the Louisiana delta, and does not believe that the interests of New Orleans will be seriously threatened by encroaching floods, yet it seems desirable to make such further investigations as may be needed to reach definite conclusions as to means and extent of possible flood reduction in order to set at rest the fears which now appear to exist among the people of the delta.

A safety valve for temporary relief consisting of a movable dam forming part of the controlling levee line, to be lowered only under conditions of extraordinary emergency, might possibly be devised, which would obviate to a considerable extent the difficulties incident to a depleted channel. A prepared floodway to control and guide the outflow would, of course, be necessary.

But the proposition to permanently lower the flood line and consequently the ultimate grade line of the levees, by means of spillways or waste weirs, should be very carefully considered before it is attempted, as past experience and the opinions expressed by many competent observers indicate that disappointment and failure must be expected.

Since the Mississippi River Commission has been quoted at length, it may be well to give some account of that organization for the purpose of judging how much weight should be given to its conclusions.

The Commission, created by Act of Congress in 1879, consists of four civilians and three officers of the Corps of Engineers, U. S. Army. Since its organization, thirty-four years ago, fourteen civilians and sixteen of the most efficient officers of the Engineer Corps have served as members; among them, Q. A. Gillmore, C. B. Comstock, and C. R. Suter. In the list of civilian members are found such well-known names as Jas. B. Eads, Henry Flad, Henry Mitchell, B. M. Harrod, Benj. Harrison, R. S. Taylor, and others familiar through long experience with Mississippi River problems.

The physical investigations and studies of the regimen of the

Mississippi River that have been made by the Commission are much more exhaustive and thorough than have ever been applied on any other river, and the deductions from this large mass of data cover practically every phase of the regimen.

The scope of the investigations which have occupied a long period of years and still continue, is outlined in the language of the law creating the Commission, viz: "It shall be the duty of the Commission to make such surveys, examinations and investigations, topographical, hydrographical and hydrometrical, of said river and its tributaries as may be deemed necessary."

The conclusions reached by such an exceptional body of able men, based as they are on a study of established facts, are entitled to the fullest confidence of all who are interested in the development and protection of the alluvial valley of the Mississippi River.

[There will be printed in an early number of PROFESSIONAL MEMOIRS an appendix to the above paper, prepared after extended research by its author, entitled "A compilation of experiences and conclusions of eminent authorities as to the utility of outlets in reducing the height of floods, citing all opinions found, whether for or against."—ED.]

El Puente de España, 1626-1914.[1]

BY

Mr. W. C. BUNNEL,
Assistant Engineer; Assoc. Am. Soc. C. E.

Spanning the Pasig River, in the city of Manila, is a bridge of great age; the original structure which occupied the same site had its beginning in about the year 1629.

From the translations of official documents as contained in Vol. XXII of Blair and Robertson, "The Philippine Islands," it appears that in 1626 the Governor, Don Fernando de Silva, reported that the city had decided to build a stone bridge over the main river[2] and that work had been started.

In 1629, Gov. Don Juan Niño de Tabora reported that the work was at such a stage of completion that the bridge could be used and would be entirely completed in a short time, and that it had been built without cost to the treasury, as the Sangleys (Chinese) had paid for it as it relieved them of ferry-boat charges.

The above books record that the work of construction was directed by the Recollect Fray Lucas de Jesus Maria, and that the bridge as built consisted of stone piers and a wooden superstructure; this class of superstructure, besides being more easily and cheaply built, had a military value in that communications between the two parts of the city, as divided by the river, could be easily and quickly destroyed by destroying the woodwork. This was done during an insurrection of the Chinese in 1638.

A historical sketch of this bridge appearing last September in a local newspaper[3] credits this work to an Augustinian friar, Antonio Herrera, and further says that this friar was the first to build arches of stone in the Orient. In order to expose the river bed, he diverted half of the water of the river into the moats and

[1] The Bridge of Spain.

[2] So called, probably, to distinguish the Pasig River from the numerous tidal channels (esteros) which intersected the site of the city.

[3] *Consolidacion Nacional.*

esteros which existed along the east and south fronts of the walled city and then cut a channel from the west end of the moat, on the south (near the present luneta) to the sea. By building heavy dikes he unwatered a part of the river bed, which permitted the construction of the piers for half of the bridge; the same plan being followed for the piers of the other half. The piers were built of a local stone, known locally as Guadalupe[4] stone or dhobe, its formation being a volcanic tuff.

The map herewith (Fig. 1) shows a part of the city of Manila as divided by the Pasig River, and the Puente de España is indicated thereon by an arrow. The next bridge upstream is now called the Santa Cruz Bridge, and is of the usual steel construction, and has been built since American occupation. The third bridge upstream is the old suspension bridge (Puente Colgante) and was built during the administration of Gov. Don Antonio de Urbiztondo de Eguia, 1850-1853. This is a very light structure and was, up to a few years ago, a toll bridge, but has lately been purchased by the city and, as a matter of safety, has now been closed to all traffic except for pedestrians. The fourth bridge is known as the Ayala Bridge. Formerly, as shown by Fig. 2, it was built with an angle in order to take advantage of the island[5] near the center of the river. It was a steel structure and was opened to traffic in 1880. In 1905, this bridge was replaced by a new steel structure of two spans, built in a straight line across the river. It may thus be seen that the Puente de España was for a long time[6] the only bridge in Manila over the Pasig River, and even up to September 2, 1914, it was carrying by far more traffic than any other one bridge. Its north approach terminating as it does at practically the center of the retail district and close to the banking and wholesale centers, has always caused this bridge to be used more than any other one.

The earliest view of this bridge that could be obtained by the writer is shown by the accompanying photographic copy[7] of an old print (Fig. 3), with a translation of the legend appearing therewith. It appears that the wooden superstructure heretofore

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[4]The name of a town about 4½ miles from Manila where this class of stone is quarried.

[5]Isla de La Convalecencia.

[6]About two hundred and twenty years.

[7]Copied from engraving in Military Information Division, Philippine Department, U. S. Army.

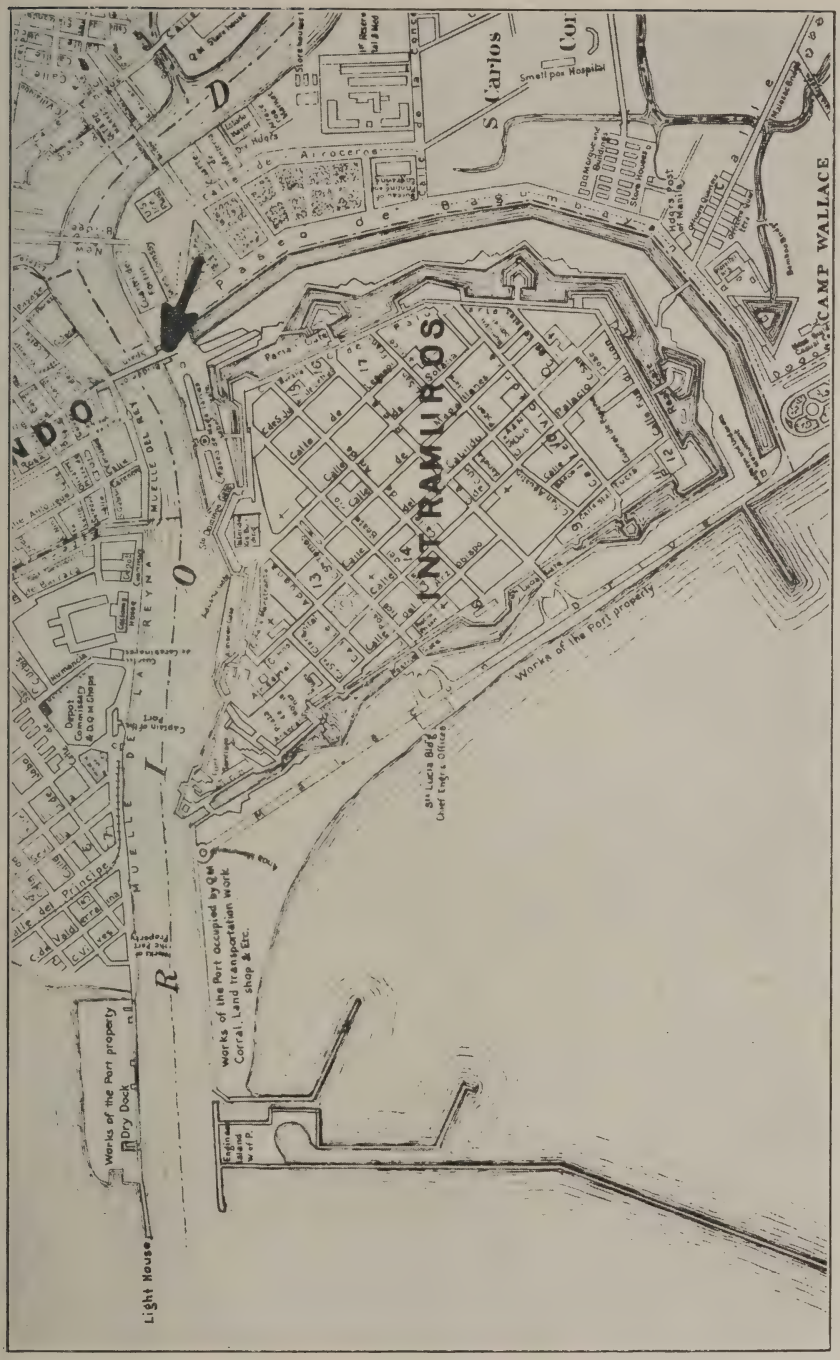

Fig. 1. General Map of Manila. (Intramuros and Vicinity.)

mentioned had been maintained up to 1814 when the bridge was rebuilt as a series of arches, and the piers strengthened. The width was about 8 varas[8] and its length about 149 varas. There is seen at the south approach a structure marked *Fortin y Mira* (small fort and lookout). Mention is made of this small fort in the early history of the defenses of Manila, its use being to guard this approach to the city.

Since the arches were built, trouble has been experienced on several occasions with the center piers; the first time was in 1824 and was due to a severe earthquake, the damage done being repaired by the engineer, Don Yldefonso de Aragon.

Fig. 2. Ayala Bridge over Pasig River, Manila, as Originally Built.

On June 3, 1863, occurred the most disastrous earthquake recorded in the history of the Philippine Islands. The bridge was severely damaged; the two central spans sank and it was thought they were going to fall into the river, and the bridge was accordingly closed. To provide a temporary means of crossing the river a ponton[9] bridge was thrown across just below the site of the Puente de España.

On examination, it was found by the engineer, Sr. D. Castro

[8]Taking the vara as 2.78 feet, the width was about 22¼ feet and the length 414¼ feet.

[9]Cascos were used for the pontons.

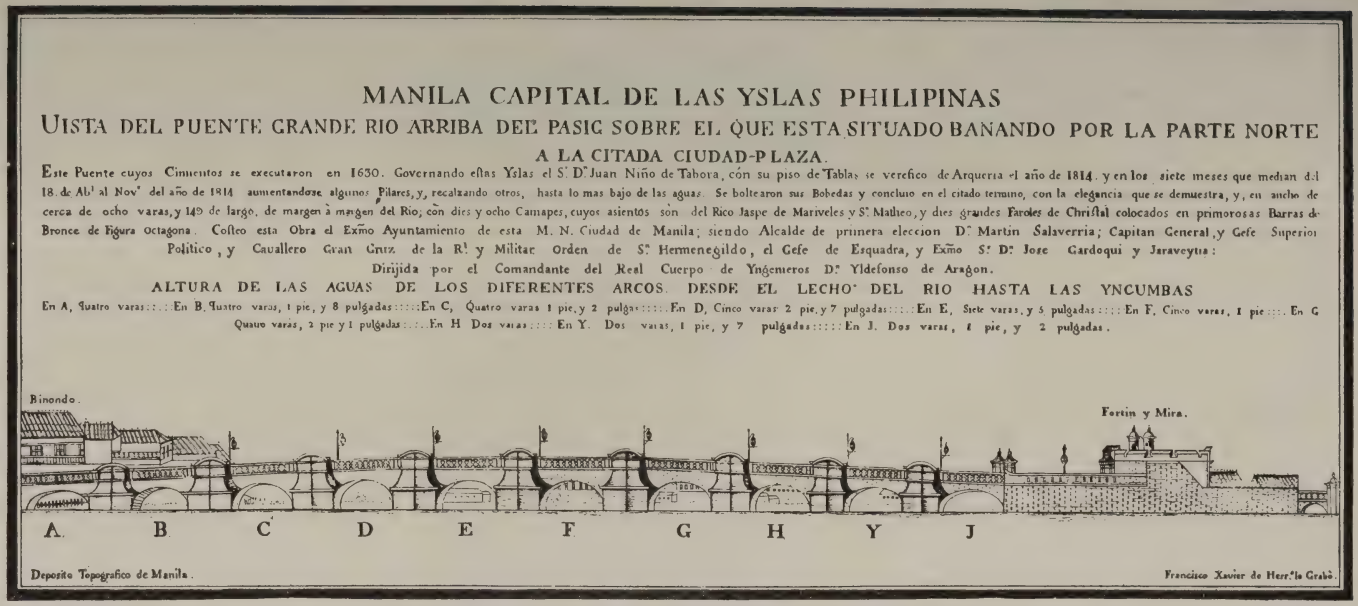
MANILA CAPITAL DE LAS YSLAS PHILIPINAS
UISTA DEL PUENTE GRANDE RIO ARRIBA DEL PASIG SOBRE EL QUE ESTA SITUADO BANANDO POR LA PARTE NORTE
A LA CITADA CIUDAD-PLAZA.
Este Puente cuyos Cimientos se executaron en 1630. Governando estas Yslas el S.r D.n Juan Niño de Tabora, cón su piso de Tablas se verefico de Arqueria el año de 1814. y en los siete meses que mediaron del 18. de Ab.l al Nov.e del año de 1814 aumentandose algunos Pilares, y, recalzando otros, hasta lo mas bajo de las aguas. Se boltearon sus Bobedas y concluio en el citado termino, con la elegancia que se demuestra, y, en ancho de cerca de ocho varas, y 149 de largo, de margen à margen del Rio; cón dies y ocho Camapes, cuyos asientos són del Rico Jaspe de Mariveles y S.n Matheo, y dies grandes Faroles de Christal colocados en primorosas Barras de Bronce de Figura Octagona. Costeo esta Obra el Exmo Ayuntamiento de esta M. N. Ciudad de Manila; siendo Alcalde de primera eleccion D.n Martin Salaverria; Capitan General, y Gefe Superior Politico, y Cavallero Gran Cruz de la R.l y Militar Orden de S.n Hermenegildo, el Gefe de Esquadra, y Exmo S.r D.n Jose Gardoqui y Jaraveytia:
Dirijida por el Comandante del Real Cuerpo de Yngenieros D.n Yldefonso de Aragon.
ALTURA DE LAS AGUAS DE LOS DIFERENTES ARCOS. DESDE EL LECHO DEL RIO HASTA LAS YNCUMBAS
En A, Quatro varas:::::En B, Quatro varas, 1 pie, y 8 pulgadas:::::En C, Quatro varas 1 pie, y 2 pulgas:::::En D, Cinco varas 2 pie, y 7 pulgadas:::::En E, Siete varas, y 5 pulgadas:::::En F, Cinco varas, 1 pie:::::En G Quatro varas, 2 pie y 1 pulgadas:::::En H Dos varas:::::En Y. Dos varas, 1 pie, y 7 pulgadas:::::En J. Dos varas, 1 pie, y 2 pulgadas.
Binondo.
Fortin y Mira.
A
B
C
D
E
F
G
H
Y
J
Deposito Topografico de Manila.
Francisco Xavier de Herr.a lo Grabò.

Fig. 3. El Puente Grande (now known as El Puente de Espana) in 1814.

Translation of legend: Manila, Capital of the Philippine Islands. A view looking up the Pasig River, of the large bridge spanning it; (the river) bathing the north portion of the fortified city.

This bridge, whose foundations were built in 1630, Sr. Don Juan Niño de Tabora governing these Islands, with its floor of boards, was rebuilt as a series of arches in the year 1814, and within the seven months intervening between the 18th of April to November of the year 1814, some piers were increased in size and the foundations of others strengthened below the water. Its arches were built and the work finished in the time mentioned, with the elegance which is shown, and, of a width of about 8 varas and a length of 149 (varas) from bank to bank of the river with 18 cappings (camapes*) whose bases are of rich Mariveles and San Mateo jasper and 10 great glass lamps placed on elegant octagonal bronze posts. The most excellent Ayuntamiento of the most noble city of Manila paid the cost of this work. Don Martin Salaverria being Mayor on his first election. (Governing); the excellent Sr. Don José Gardoqui y Jaraveytia, Captain General and superior political chief and knight of the grand cross of the Royal and Military order of Saint Hermenegildo and the chief of the squadron. Directed by the Commandante of the Royal Corps of Engineers, Don Yldefonso de Aragon.

Height of the water at the center of the arches, from the bed of the river: In A, 4 varas; in B, 4 varas, 1 foot and 8 inches; in C, 4 varas, 1 foot and 2 inches; in D, 5 varas, 2 feet and 7 inches; in E, 7 varas and 5 inches; in F, 5 varas and 1 foot; in G, 4 varas, 2 feet and 1 inch; in H, 2 varas; in I, 2 varas, 1 foot and 7 inches; in J, 2 varas, 1 foot and 2 inches.

*This word can not be found in any dictionary. It is taken to mean the caps finishing the tops of the piers.

Olano, that the buttresses and a sufficient number of the piers were in a satisfactory condition to permit repairs to be made. It was now that the two central arched spans made up of steel ribs were put in place, and other work done to strengthen the structure. Each of these spans had five steel ribs, details of which may be seen in some of the accompanying photographs. It will be noted that Fig. 3 shows ten arches,[10] while all views of the bridge as it was rebuilt in 1875 show eight arches, which shows that two of the piers near the center of the river were removed. This can further be seen by noting that the spans of the steel arches are

Fig. 4. View of Bridge of Spain Prior to Work Done in 1901.

twice those of the stone arches. This work was completed in 1874, and on January 1, 1875, the bridge was formally opened to the public and given the name of El Puente de España, the name by which it is known to-day. This name was officially adopted on January 29, 1875. Prior to 1875 the bridge was known by two names, El Puente de Piedra (bridge of stone) and El Puente Grande (great bridge).

The bridge as completed in 1875 remained as then built up to

[10]An English writer in 1822 mentions this bridge as follows: "over the river is a neat but narrow stone bridge of ten arches."

1901, when it was widened by the addition of the present sidewalks, this work being done under the supervision of, then, Lieut. William D. Connor, Corps of Engineers.

Fig. 4[11] is a view of the bridge prior to the addition of the sidewalks, and as it was up to the time of American occupation. By comparing Fig. 4 with Fig. 5 the work added in 1901 may be seen.

During the flood of 1904, which surpassed that of September, 1914, the bridge was damaged by drift and lighters, and a part of the base of one of the piers was scoured away. A cofferdam was built around this pier and repairs made, and riprap placed around the other piers.

Fig. 5. View of Bridge of Spain After the Work of 1901.

In 1905, the bridge was double-tracked for the electric street cars[12] and the loads and the number of them increased enormously as auto-trucks superseded carabao carts.

Just below the bridge, the river has, since 1905, been considerably restricted by the construction of the river wall[13] along the south bank, and a great deal of dredging done just below the bridge, the project being to maintain a depth of 18 feet from the bridge to the

[11]Copied from W. D. Report, 1899.

[12]Some of these cars weigh 40 tons.

[13]This wall is not shown on Fig. 1. At the time this map was made the water reached some of the bastions and part of the curtain of the north front of the walled city.

bay. Manila is built on a low, flat area intersected by numerous tidal channels, many of which have been closed. When the flood of September, 1914, came down there was not the former opportunity for dispersion, and it swept down the river with increased velocity and volume.[14] The piers being built, below water surface, of a stone that does not offer great resistance to erosion and having been exposed to the current for so long, the center pier failed during the flood.

This flood occurred during the season of rains and storms; the rains causing the flood began several days before, with the maximum rainfall occurring in the twenty-four hours ending at 7 p. m. September 2, when it reached 9.72 inches. Shortly before 11 o'clock on the night of September 2 it was discovered that the

Fig. 6. The Flood of September, 1914.

center pier had sunk to such an extent as to cause alarm as to the safety of the structure and the bridge was closed to all traffic. Fig. 6 shows the rush of water through one of the arches near the shore.

The pier continued to settle slowly, and work was in progress removing the superstructure composed of the two steel arch spans, when, at about 2 o'clock on the afternoon of September 14, with a loud crash, the pier split vertically at about the third point from

[14]From the city records it was learned that the slope of the river during the flood, from the bridge to the mouth, was about 6 inches, and the slope in the region from the Ayala Bridge to the Puente de España was about 24 inches. The flood discharge was 43,200 cubic feet per second. The piers obstruct about 25 per cent of the channel.

the downstream end, and the upstream end settled rapidly until the haunches of the outer arch ribs were at the water surface. Figs. 7 and 8 show conditions soon after the crash. It is reported that the pier was struck by a launch just before it split. The removal of the two steel spans was then carried on vigorously, all the flooring system being removed and the arched ribs lifted with a large floating derrick, after which they were placed on shore.

Fig. 9 (looking towards the north) shows both spans removed. From this view and Fig. 10 the nature of the failure of the pier may be seen. Fig. 11 shows one of the steel ribs being hoisted. Some idea of the floor system may be obtained from Fig. 12, and a

Fig. 7. View Shortly After Failure of September 14, 1914.

general view of the wrecking operations is shown by Fig. 13.

Due to the swift current and turbidity of the water, it was not possible to start diving operations to determine the condition of the remaining piers until September 13, and but little work could be done at that time. The examination was begun again on September 15 and completed on October 7, but as soon as the examination had progressed far enough to show that the piers on either side of the demolished one were safe to carry a temporary structure, plans were started for a wooden Howe truss to span the opening.

The condition of the underwater work as determined by the diver

was, briefly, as follows:[15] First pier out from the north end—the south side of the footing of this pier is badly eroded and contains several deep holes; the upstream end could not be examined, due to the presence of submerged driftwood; the north side of pier was found to be more eroded than the south side; the downstream end was found to be in good condition; all the masonry above the footing was in good condition, being protected by a granite facing. Second pier from north end—this pier was found to be in poorer condition than the others, but this was expected as the force of the current is stronger on the north than the south side. Third pier from north end—the footing of this pier was found to be badly

Fig. 8. View Shortly After Failure of September 14, 1914.

eroded on the upstream end, and the north side with large holes extending under the footing. This footing extends about 22.9 feet from the face of the pier out into the stream with only about 2.5 feet of water over it at low tide. Third pier out from the south end—the footing of this pier was found to be dangerously eroded on the south side and upstream end; the riprap around it was

[15]A great deal of the information contained in this article has been taken from brief historical notices appearing from time to time in the daily newspapers of Manila, and the details with respect to the work completed in December, 1914, were kindly furnished by city officials connected with the work.

Fig. 9. After Removal of Both Steel Spans, Showing Condition of Center Pier.

Fig. 10. Conditions on September 20, 1914.

almost entirely scoured away and many of the blocks have been undermined and have fallen out; the north side could not be examined owing to interference with navigation, but soundings indicated that this side of the footing was still protected by riprap. It is understood that some protective work is contemplated before the next storm season.

The demolished pier was found to consist of a concrete made of Guadalupe rock and faced with excellent granite to a short distance below the water surface, and all the masonry below this was of the Guadalupe rock.

On October 10 the Municipal Board approved the recommendations of the city engineer as follows: "that a temporary wooden

Fig. 11. One of the Steel Ribs.

Howe truss bridge be built to span the present opening between piers 3[16] and 5[16]. This span should have three trusses with two 4.15 meter clear roadways and two sidewalks outside the trusses. It should be designed to carry the following classes of traffic: carromatas and carriages, bull carts, automobiles and auto-trucks up to 8 tons capacity and foot traffic."

The structure as first designed was for 3½-ton trucks, but it was found that with a little increase in cost an 8-ton capacity bridge could be built.

Proposals were then invited for the work; the lowest bid re-

[16]Counting from the north end.

Fig. 12. View Showing Floor System.

Fig. 13. Wrecking the South Steel Span.

ceived being $19,000.00, and the contract to be entered into required the completion of the work within seventy days.

Believing that both the cost and time were far in excess of what they should be for this work, the city decided to do the work itself.

Accordingly, work was started as soon as possible, and on December 15 the span was successfully drawn across the channel and rested on the piers.

The span was built complete on the southern part of the bridge remaining after the removal of the two center spans. When the truss was completed it was raised, and a number of pairs of car wheels placed underneath; it was then run forward until about 50 feet projected over the channel; a large barge was then placed underneath this overhang with about 10 feet on the truss extending beyond the barge and crib work built up to it; with a tackle across

Fig. 14. View of New Span, Completed December, 1914.

the channel and a road roller on the north approach, the span was easily and quickly drawn into place.

The new span is 176 feet 2 inches long, with two roadways, there being three trusses 15 feet 8½ inches on centers and two overhanging sidewalks, each 4 feet 8 inches. The work was completed in forty-seven days and at a cost of approximately $11,250.00, of which $500 was for plant that is now available for other use—the total amount of which should not be charged to this work. Oregon pine lumber was used for the construction of this new span. A view of the new span in place is shown in Fig. 14.

The work having been completed, the bridge was formally opened for the use of the public on the afternoon of December 22, 1914, with appropriate ceremonies, and again this old structure is carrying a great part of the traffic[17] across the Pasig River.

[17]On the first day after placing the bridge in use a total of 9,500 vehicles of all classes crossed the bridge between the hours of 8 a. m. and 4.00 p. m.

The Estimate of the Situation.*

BY

Rear Admiral AUSTIN M. KNIGHT,
U. S. Navy; President U. S. Naval War College.

If we recognize that the real work of a war college is training for war command, we shall be led to inquire at what ends this training should aim, and by what methods it should proceed. Evidently it must aim first of all at the development of logical and concentrated processes of thought, which, beginning with a clear recognition of the end to be attained and the facilities and difficulties involved in the situation, proceeds by logical steps to a decision upon the course best fitted to attain the end.

There is nothing here that is peculiar to training for war command as distinguished from other training which looks to the development of intellectual processes leading to definite action.

Whenever we find ourselves confronted by a situation which calls for something to be done, we pass from recognition of the necessity for action to the action itself by mental processes which, often without deliberate consciousness on our part, follow a certain clearly defined course. We see the something to be accomplished, evaluate and balance the factors entering into its accomplishment, and decide upon the way of going about it. In many, perhaps in most, cases, the something to be done is rather vaguely seen, the evaluation of factors involved is incomplete, and the decision is hasty; but the process, however superficial, is inevitably logical to the extent that some sort of a decision precedes the action, some sort of an estimate precedes the decision, and some recognition of the end to be attained precedes the estimate.

Where great issues depend upon the action taken, as is the case in war and the preparation for war, it is of vital importance that the estimate should be carefully made, covering all phases of the situation, giving adequate recognition to facilities and to obstacles, adjusting means to ends, and binding the something to be done with the decision how to do it, by a thorough and logical process of reasoning. This we call "Estimating the Situation."

The "estimate of the situation," then, in its military application, differs from the ordinary mental processes which govern the

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acts of our every-day life only in this—that it is a very thorough and a very methodical course of reasoning, directed along lines carefully systematized with a view to giving the fullest attainable assurance that no important factor shall be overlooked, and guarded from hasty and superficial treatment by the formality which is deliberately imparted to it in the teachings of the War College. It is a logical process which, starting with a mission to be accomplished and taking account of all existing conditions, leads up to a decision which, when reached, is seen to have resulted more or less inevitably from the mission and the conditions considered together.

The form which has been adopted at the War College for estimating the situation follows in its general features the forms which have been developed and employed by such authorities as Buddecke, Grippenkerl, and Audibert. There are differences in detail between these authorities, but all are agreed as to the necessity of providing a form which shall serve as a guide; and the differences in the forms which they suggest are such as to accentuate the importance of the general principles which lie at the foundation of them all. It is recognized, of course, that a form is an empty thing in itself, and that too much stress might easily be placed upon it. But experience has shown that it is always helpful as a guide to logical processes of reasoning, and that to a beginner, at least in the field which we are now considering, it is altogether indispensable. It is not to be supposed that in actual warfare every situation which arises can be dealt with in detail and in writing, but the habits of thought which are developed by the practice of this system in cases to which it is applicable—as in the problems of the War College and in most of the strategical situations of actual war—should result in an instinctive application of the same principles when a sudden emergency calls for an instantaneous decision. Our insistence, then, upon adherence to a prescribed form is a means to the end which has been laid down as that toward which the training for war command should first of all be directed—the development of logical and concentrated *habits* of thought.

In the problems which are drawn up at this and other war colleges, an effort is made to simulate conditions as they would confront a commander operating in the field. At many points it is easy to do this; at others it is so difficult that only partial success can be hoped for; at still others it is altogether out of the question. One of the most important points at which this effort fails is in the initial survey of the conditions upon which our estimate of the situation must be based. In practice, a commander must make this survey for himself, outlining the field of operations in such a way as to secure a broad outlook over the field with all factors in their proper relation to each other. In war college problems the initial conditions are necessarily stated by the framer of the problem. This difference between theory and practice should be kept in mind by the student who must remember

that, in actual warfare, his estimate of the situation should be preceded by an outline sketch of the situation as a point of departure for the estimate. This preliminary sketch will closely resemble the *statement* of a problem issued for study at the War College.

In the form used at the War College the estimate of the situation is treated under four heads, as follows:

1. The Mission;
2. The Enemy Forces: Their Strength, Disposition and Probable Intentions;
3. Our Own Forces: Their Strength, Disposition, and the Courses Open to Us;
4. The Decision.

The mission is the task which lies before us for accomplishment, and a clear understanding of this task is the first step toward its accomplishment. How shall we arrive at such an understanding? That depends upon conditions. If we are acting under orders received from higher authority we must begin by studying these. If they have been properly framed they will themselves define the mission, and may, if clearly and concisely expressed, be accepted at once as a statement of the mission, subject, however, to modification in the light of *new* information which may have come to the knowledge of the officer receiving the orders and which was not in the possession of the higher authority when the orders were issued. The principles governing in this case will be explained hereafter. If the form of the orders does not lend itself directly to this application, we must search out the essentials and throw these into a shape as concise and clear as is practicable. If the instructions are too general to be regarded as defining a mission, we must apply their spirit to the conditions which we find existing and deduce a mission for ourselves in harmony with this spirit.

In the extreme case, we may have no instructions and no source to which we can look for them. Here we must be guided by a broad conception of the duty which our position carries with it, and the responsibilities of loyalty and initiative which result from it.

We may say, then, that in general, to determine a mission based upon orders, we study the orders; while to determine one based upon a situation, we study the situation.

A variation sometimes introduced in the form is to consider "Our Own Forces" before "The Enemy Forces." This has some advantage in cases where the mission is primarily offensive. It is a matter of detail, chiefly important because of a tendency which we may assume as inherent in it, to fix the attention upon offensive rather than defensive action, and to encourage *initiative* in the *mission* and the decision.

The mission is always the ultimate task which *at present is*

assigned to us for accomplishment—the end at which we are told to aim and toward which everything included *in our present estimate* must be directed. There may be, and often will be, many steps by which we advance toward this end; and it may be that only one such step demands immediate action and immediate instructions to our subordinates. Each such step, directed along a line which leads toward the end at which we are aiming, constitutes in itself a secondary or temporary mission, which may become, and usually does become, *the* mission of a subordinate commander. These secondary missions must not be confused by the commander-in-chief with the mission upon which his attention is to be fixed, and upon which all of his efforts must converge. This may be "To Destroy the Enemy's Army," "To Capture the Enemy's Capital," "To Destroy the Enemy's Fleet."

It may be that the first step toward the capture of the enemy's capital is to gain possession of a certain mountain pass, and this may be a task assigned to a subordinate commander, for whom it becomes *the* mission. It may happen that this preliminary operation calls for all of the forces available, and that it must be directed by the commander-in-chief himself. This does not make the capture of the pass "the mission" for him, if, as in the case assumed, his instructions reach beyond this to the capture of the capital. Nor is the situation changed even if he can not at once decide what steps must follow the capture of the pass. So long as there remains before him under his instructions a larger task to which the task immediately at hand is only one step, he must take as his mission the larger task which lies at the end of his road.

This brings us to a point which has been much debated, and about which there has long existed a difference of opinion at the War College and elsewhere. This difference, as it happens, is a matter of form rather than of principle. It is well illustrated in the situation which has just been outlined. A commander, acting under instructions to capture the enemy's capital, finds that the first step before him is the capture of a certain mountain pass, and that this operation calls for the employment of all his forces and must be directed by himself.

If the task were assigned to a subordinate commander, it would manifestly be the mission for that commander. Is it, then, the mission of the commander-in-chief, since he is personally to direct it?

The answer is that it is not for him *the mission*, but assuredly it is *a* mission, and one for which he must prepare by an estimate which stops with the capture of the pass. It in no sense replaces or obscures *the* mission toward which it is only the first step, but for the moment it demands an estimate and a decision of its own.

We decide, then, to recognize in certain problems, an *immediate mission* leading up to the accomplishment of the general mission. This immediate mission may become apparent in our study of the general situation, in which case it will appear as a part of our

general decision. In the case which has just been considered, the commander-in-chief, in estimating the general situation, finds that a necessary step toward the accomplishment of his general mission—the capture of the enemy's capital—is the capture of the mountain pass, and the general decision includes, we may imagine, some such phrase as this: "Proceed to capture the ——— Pass, and then deploy beyond the mountains, advancing in three parallel columns by the three main roads leading across the valley, investigating the city on the east, west and south, and sending the cavalry to cut the communications from the north." Here is the general decision, and as a part of this we find an immediate mission—to capture the pass—as a step toward the accomplishment of the general mission.

The commander-in-chief has, in effect, assigned himself a new mission, included in the larger one, exactly as, under other conditions, he might have assigned the smaller mission to a subordinate commander. The new mission *calls for a new estimate and a new decision,* all keeping in view the larger mission and looking toward its accomplishment. This means that each individual step must be so planned that it shall lead up to, and prepare the ground for, the step which is to follow. If, for example, the first step is to locate the enemy's raiding force and the second to destroy it, it will be futile to locate the force without making provision for the step which is to follow.

In what precedes we have seen an immediate mission developing in our estimate from conditions known or assumed to exist; but there are other cases in which such a mission develops unexpectedly from the progress of events. In working toward the general mission along the lines of the general decision, a situation arises which was not foreseen when the decision was framed. Here, again, the new situation must be estimated upon, and a new decision reached. We shall see when we come to discuss the relation of the *mission* to *orders* what principles should guide us here.

An immediate mission, whether it grows out of our general estimate or forces itself upon us suddenly as a result of new conditions, demands immediate attention and draws to itself the whole of our loyalty—not in any sense withdrawing this loyalty from the general mission, but presenting a new channel through which it flows. We concentrate all our resources, then, upon the accomplishment of this mission, directing our efforts along the lines of the decision which develops from the immediate mission.

We have seen the importance of guarding against a too narrow limitation of our mission. But we must remember that it is of not less importance to guard against projecting it too far. In the usual case, where we are acting under instructions, these instructions will define the bounds within which we are to act. The task assigned to us will usually be only a part of the larger task included in the mission of the higher authority—one of many factors in the decision under which he is acting toward the accomplishment

of his mission. Included within the larger purposes at which he aims, and embraced within the wider field with which they deal, there may be other tasks for us, but their assignment and their definition lie within his area of responsibility and outside of ours. It will be his duty, at the proper time, to coordinate those tasks with the tasks assigned to his other subordinates so that all shall be directed toward the accomplishment of his more inclusive mission.

Each subordinate commander takes the task assigned him as his mission, estimates the situation, reaches a decision as to his course of action, and hands down new tasks to his subordinates, each one of whom, in turn, takes the task assigned him as his mission, and so on indefinitely.

It will be seen from what precedes that just as the mission for any commanding officer flows directly from the orders which he receives, so the orders which he issues to his subordinates flow in turn directly from his mission. Thus the mission is linked up on both sides to orders—flowing from orders received and dictating orders issued.

It is important that this relation between *mission* and *orders* be thoroughly grasped. Every military situation incident to a state of war derives its importance from its relation to the war as a whole. The most successful conduct of war requires that each act of war be directed towards the attainment of the object of the war. War is not simply fighting to injure the enemy; it is fighting for the attainment of a definite purpose. And since each act of war should be in harmony with the general purpose, it is evident that there must be a supreme control to direct all of these acts. This control — in the field — is exercised by the commander-in-chief. He is cognizant of the general purpose. His plan assigns tasks to forces. The success of his plan depends upon the intelligence with which his subordinates execute the portions of the plan allotted to them. If they, on their own initiative and without necessity, change their parts of the plan, the commander-in-chief then is no longer working with a plan that is a *whole*, but with several plans of parts. The plan of the *part* may succeed, but the success may be worthless because it does not fit into the plan of the whole. These considerations make it clear why the *mission* must normally be derived from the orders, and indicate the great importance of writing orders so clearly that they can not be misunderstood, and so comprehensively that no important detail shall be omitted. The officer who receives an order, since it is to be the guide to recognition of his mission, should find in it as nearly as possible all that he needs for such guidance. It should as far as possible acquaint him with all conditions which are necessary for an intelligent comprehension of the situation, making clear to him especially the information upon which it is based and the end toward which it is directed; that is to say, the end which the higher authority has in view. Being thus placed as nearly as

possible at the point of view of the higher authority, the officer receiving the order is in a position to accept his mission with that combination of loyalty and initiative which is the ideal of obedience.

We shall see hereafter (in considering the Order Form) that the information above referred to as essential is included in paragraphs 1 and 2 of the form for Order-writing which has been adopted as standard; and that paragraph 3 of the same order contains the definite instructions handed down by higher to lower authority.

There are, of course, many conditions under which orders can not be treated with ideal fullness and formality; but every order, however brief, whether written or verbal, must—to be an *order* in the military sense of the word—contain the essentials of these paragraphs; that is to say, it must accompany the *command* of paragraph 3 (Order Form) with a statement of the object to be attained by the order (paragraph 2), and a statement of the *information* upon which the order is based (paragraph 1).

It may happen that the conditions upon which the order was based have changed since it was issued, so that a modification or even an entire change of plan is required on the part of the officer receiving it, *to make his action contribute to the plan which he knows to be that of the higher authority.* The new situation will point to a new mission, demand a new estimate, and dictate a new decision. The new mission may replace the earlier one, or postpone its execution, or merely introduce a new step leading up to it.

An officer finding himself in this position will, of course, communicate with his superior if circumstances permit, reporting the new conditions and requesting new instructions; but if this is not practicable, he will modify his mission as may be necessary, *always in the direction of the end at which he knows his superior to be aiming.* For this, his orders, if properly framed, will constitute at once his justification and his guide. If he has no knowledge of the plans of his superior, so far as these apply to a situation suddenly confronting him, he may have such an intimate acquaintance with the views of his superior and of the principles which make up the code of that superior, that he will act instinctively in accordance with these principles. A code of this character definitely held by a superior and so fully absorbed by his subordinates that it stands for them in the place of orders, is called—for want of a better term—a "Doctrine." The subordinate who acts as thus described is said to be "indoctrinated."

There is nothing so important in war as obedience, but this does not mean mere mechanical obedience. Ask first, "What was my superior's intention regarding this order? What may have been his view on this subject? What information did he have when the order was issued? What additional information have I now? How would this information have affected my superior's order?"

It is only thus that dead mechanical obedience is avoided and active obedience combined with initiative is produced.

* * * * * * *

The mission, having been decided upon, is written out clearly and concisely and forms the first paragraph of "The Estimate of the Situation."

The following are typical examples of the mission:

Mission.—To secure command of the sea.
Mission.—To capture or destroy the enemy's fleet.
Mission.—To detect and report any movement of the enemy main body from Culebra.
Mission.—To prevent enemy force from throwing reenforcements into Guam.
Mission.—To capture or destroy enemy convoy and escort before they reach the Strait of Gibraltar.

Having decided upon the mission—*what to do*—we must study the situation from all points of view as a means of arriving at the decision—*how to do it.* This part of the estimate is considered under the following headings:

2. Enemy Forces: Their Strength, Disposition and Probable Intentions.
3. Our Own Forces: Their Strength, Disposition and Courses of Action Open to Us.

Under these two headings we are to consider all conditions entering into the problem of carrying out our mission, discussing fully each course which may be open to us or to the enemy, weighing advantages and disadvantages and balancing one course against another, until we are led by a logical train of reasoning to a decision upon the course to be adopted.

It is very important that the whole train of reasoning, with all the facts, beliefs, and theories upon which it is based, be fully written into the paper. This will not always be possible in war, but we are not now conducting war, we are *training* for war; and the practice of taking into consideration every detail that bears upon our problem will lead to a habit of thoroughness which will assert itself instinctively as a feature of our reasoning even when our estimate and our decision must find their only expression in action or in orders to our subordinates.

A little consideration will make it evident that the treatment of 2 and 3 above must overlap at many points. Comparisons must be made which call for bringing our own and the enemy forces together in one paragraph; and this is true not only of the opposing forces in their entirety, but of individual detachments which may be expected to oppose each other. And the *position* of one force must usually be defined by direct reference to opposing forces. Similarly, the *aims* of one force and the *courses of action* open to it can

hardly be stated without referring to the other force. It follows that, in spite of what has been said about the desirability of adhering to a standard form, we must be prepared to accept, especially at this point, a reasonable degree of elasticity in the use of the form.

The situation must be studied, as nearly as possible, from the point of view of a commander afloat. It is well to begin with a general survey of the field operations, laying out the charts of the area, indicating upon these the locations of the forces on both sides, the distances between important points, and other factors which will help toward visualizing the situation as a whole. Pilot charts, sailing directions, etc., should be studied, together with all other available sources of information. Special features will almost immediately force themselves upon the attention as demanding consideration, and to some extent defining courses of action.

If we have already studied the field in a general way as preliminary to the choice of a mission, we here go over it again more in detail and with the added light which will result from the fact that our mission is now known. We studied the field before to decide what we would do. We study it now to decide how we will do it.

2. Enemy Forces: Their Strength, Disposition and Probable Intentions.

Under this heading we deal with the difficulties which are to oppose us in the accomplishment of our mission, and among these we must include those obstacles of wind, weather and terrain which are allied with the enemy against us.

Here we are concerned, first of all, with *information*, and secondarily with *inference*. In actual war, as we all know, information is always incomplete, often inaccurate, sometimes altogether lacking, and frequently deliberately designed to deceive. It must be sifted, weighed, tested, to determine its reliability. Even if our information is full and accurate, our inferences from it may be altogether at fault.

In drawing up problems at the War College, the information is such as would probably be in the possession of an officer called upon to deal with a similar problem in actual war.

Information of the enemy is important, but an estimate of his information of us is also important. The initiative is largely a matter of information. If we know and he does not know, that is one thing; if we know and he knows too, that is quite another thing. A problem solver must, therefore, consider all he knows of the enemy and then record all of the deductions and inferences which he adds to his actual knowledge.

The reasoning should be so arranged that each step results logically from what precedes and leads up to that which follows. *The estimate is not for the purpose of justifying a decision previously arrived at. It is a reasoned solution of a problem where*

each step in the process approaches a decision which, without those steps, could be arrived at by accident only.

The effort should be, in this part of the estimate, to arrive at the enemy's point of view, to think as he would think of us; to consider all the plans that he would consider, and to estimate which of those plans would be most injurious to us. This is the most difficult part of the estimate. The intentions of the enemy are frequently not divined. Clausewitz said: "There are always only three cases possible, and when all these have been provided for, the fourth invariably happens." Wellington said: "The great thing is to know from this side of the hill what the enemy is doing on the other."

It will not be possible in most cases to arrange the appropriate reply for all possible intentions of the enemy. But all courses of the enemy should nevertheless be carefully considered, to avoid being taken by surprise. The strategist must always be ready with a remedy for a new situation, but he will rarely issue the remedy until the situation arises. One must endeavor never to be caught in a situation that has not been foreseen and considered as a possibility. It is only by a thorough and painstaking consideration of the enemy's possible intentions that surprise can be avoided. Anyone familiar with military history knows the great moral value attached to surprise.

The following points call for detailed consideration, not necessarily under individual headings, but with full recognition of their individual significance. They are given here in the hope of insuring a full and detailed statement of the course of reasoning in the *written* solution of the problem. It is not assumed that they cover all the points to be considered.

(a) Position and strength of enemy forces as far as can be inferred from information available.

(b) Elements of time and space which govern the situation and affect the enemy's possible movements, and his possible whereabouts after certain intervals of time.

(c) What will the enemy probably take as his mission?

(d) What courses of action will he probably consider as leading to the accomplishment of his mission? Discuss the advantages and disadvantages of these courses from the enemy point of view, including logistics, weather, the reply which he foresees that we may make, etc.

(e) Which of these courses will he probably adopt as his initial plan of campaign, and why?

3. Our Own Forces: Their Strength, Disposition and Courses Open to Us.

Under this heading there should be a thorough examination of all courses of action open to us which lead toward the accomplishment of the mission. All other courses, however attractive, must be discarded.

Each course of action should be clearly outlined and definitely considered. Having clearly outlined a course of action, examine it. Does it accord with the mission? What is its prospect of success? What will be the penalty of failure? Does it build for the future? Is it what the enemy will expect? What can he do to defeat it? Arrangements for the sure attainment of the *mission* are of first importance, *then*, for an attainment of the mission by methods that will bring the maximum ultimate profit.

The following are among the points which should receive attention. In whatever way they may be grouped, the written discussion should make it clear that they have all been considered:

(a) Position and strength of our own forces.

(b) Compare (a) with strength and distribution of enemy forces as already outlined, and consider the elements of time and space which enter into our own and the enemy's possible movements.

(c) In view of our mission and of the strength and disposition of the enemy forces, and his probable mission, is our task offensive, defensive, or offensive-defensive?

(d) What courses of action are open to us *as leading to the accomplishment of our mission?*

(e) How are these courses affected by the mission which we have considered as likely to be adopted by the enemy and by the initial plan which we have assumed that he will probably adopt?

(f) How will these various courses stand if the enemy's initial plan is different from what we have assumed it as likely to be?

(g) What reply can the enemy make to each of the courses outlined in (d) as open to us?

(h) What difficulties, other than those directly connected with the enemy's forces, will be encountered in the various courses outlined in (d)?

"The influence upon one's own plans exerted by the conclusions formed concerning the enemy's probable intentions varies greatly according to the situation. *A commander should endeavor always to keep the initiative*, and this can only be done by acting aggressively. In war it is better to lead than to follow suit. A commander whose mission makes offensive action desirable and whose force is greatly superior to the enemy's, clearly should not permit the enemy's movements to change his general plan of action. On the other hand, the dispositions of a commander, whose mission is strictly defensive, and whose force is greatly inferior to the enemy's, may be considerably affected by the movements of the enemy. So long as possible, however, *a commander should keep the reins in his own hands.* His mission, and not the enemy's movement, should as long as possible be considered the governing factor."—Capt. R. S. Fitch, U. S. Army.

When all reasonable courses of action have been examined and compared, one of them is selected. Thus we reach a *decision* as a result of a logical process of reasoning, which process is so fully set forth that its reasonableness is apparent not only to others who read our estimate, but to our own minds as we review it.

If the nature of our problem is such that an *immediate* mission grows out of our estimate, it is at this point that we shall recognize it. It will, in fact, appear as a part of the decision, as in the illustration already given, where a mountain pass is to be captured as the first step toward the capture of a city. In this case we proceed now to a new estimate of the situation growing out of the new mission.

The decision, then, is the plan decided upon for attaining the mission. It outlines a definite plan for the use of the forces available. The decision once reached must be unwaveringly carried out with our whole energy. Buddecke says that this unwavering execution of a decision once reached "is of such great importance in tactical affairs that the best course of action, if carried out half-heartedly, will come to naught, while a mistake in the choice of course can often be entirely offset by decided and confident action."

The *decision* must not betray indecision. The decision is the basis of orders to be written to subordinates. These orders must show resolution. "Resolution is not a heaven-sent gift. It is a quality of mind, the product of keen desire to perform a certain act, and the sure knowledge how to accomplish it."

Ingelfingen says: "Indecision on the part of the commander-in-chief may quickly destroy the powers of the troops and render in a short time the whole army unfit for battle. Hence in war it is better to undertake something with firm determination than to vacillate hither and thither, order, counterorder, disorder. In war we should adhere to what we see to be good enough and not be led astray by seeking for something better."

Dodge says in his life of Napoleon: "Many a general fails by forgetting his first and better intention, and being led astray by an unimportant gain to do what is not in natural strategic sequence. A sense of proportion, of the relative value of things, is one of the highest qualities in the captain as in every other workman."

Captain Eltinge, in his Psychology of War says: "The leader whose ideas are not clearly defined and whose intention is vacillating will get only half-hearted action from his troops, while, on the other hand, a determined man who has one clear idea will himself be surprised to see how the troops respond."

Grippenkerle says: "*Arrive at a definite decision.* . . . I warn you against half-measures. For example, if you have decided to attack, do so with all your troops; if you wish to retreat, do not come to a halt again after a few miles without the weightiest reasons. *Be perfectly clear in your own mind what you want to do*, and then carry it out to the letter."

In the solution of problems at the War College, as in the actual

practice of war, the need of a ground work of sound military knowledge is constantly experienced. If it happens that a strategic problem is presented, there is a demand for knowledge of strategic principles. If it is a tactical problem, then tactical knowledge and experience are required. It is quite natural for a beginner to feel that there should be some guide to his first efforts. So far as *form* is concerned, this demand is reasonable, and is easily met by the issue of a few simple problems with solutions in correct form. More than this the college can not offer in the shape of definite instructions. It has no code by which naval warfare is to be conducted. It presents no rule for meeting any single one of the situations which may arise during the progress of a war. It would manifestly be impossible to formulate such rules, for the reason that the problems which arise in practice are infinitely varied in the character and significance of the factors which enter into them. This is true not only of conditions which have to do with the material forces involved and the features of wind and weather and terrain, but with the intellectual and moral forces of the personnel. Thus each problem must be dealt with on its merits, in accordance, however, with certain general principles, the recognition of which is one of the objects of our studies. The college aims, then, to furnish, not rules, but principles, and to develop a habit of applying these principles logically to each new situation as it arises. It offers not a set of precepts, but a course of training. It presents for study a series of problems illustrating situations such as might arise in practice, and which are regarded as involving instructive principles of strategy and tactics. These problems are studied carefully, and each officer presents what, for want of a better term, is called a *solution;* but it must always be recognized that neither the result thus obtained nor that obtained by maneuvering on the chart or on the tactical board is a "solution" in any sense of finality. For every problem of this character which can be devised, there are many possible solutions, and no one of them is so good that we can safely say that it is the best possible. In a large number of proposed solutions, however, we shall recognize many degrees of excellence, and it will sometimes happen that one of these will stand out as manifestly the best of those *under consideration.* Whatever their degree of excellence may be, all are acceptable if they start from sound premises and proceed by logical reasoning to a sound decision.

Each problem given to the class is considered very carefully by one or more members of the staff, and a staff solution is prepared. This differs from the solution prepared by members of the class only in the fact that it is usually the work of an officer of considerable experience, having the advantages of consultation with other members of the staff, all of whom are supposed to be "indoctrinated" with principles which are believed at the War College to be sound. All that is claimed, therefore, for the staff solution of any problem is that it is a sound, or, as it is more commonly called, an "acceptable" solution—not that it is necessarily better than all or any of those among the class solutions, which are found to be also acceptable.

The following tactical problem, with solution, illustrates in a brief form the general character of the estimate. Strategic problems, covering the ground much more fully, are issued by the War College to officers in attendance on its course, and to those taking the correspondence course.

TACTICAL PROBLEM.

Blue Situation.

Motives.—Estimating tactical situations. Attack on convoy under escort.

General Situation.—War exists between Blue and Red. A Blue force of 4AAs is operating to intercept and destroy a Red convoy that is known to be proceeding to a base 100 miles eastward of Blue's position at 8 a. m. Blue knows that Red has at the base a force of AAs and Vs, the AAs of inferior speed to Blue AAs. Blue also knows that the escort for the Red convoy is composed of BBs. Red BBs are inferior to Blue AAs in gun power, but are superior in speed and torpedo power.

Special Situation.—At 10 a. m., Blue, steaming east in column at speed of 21 knots, sights smoke bearing SSE, distant about 15 miles, and on investigation finds this to be an enemy force of 5 BBs. At the same time, Blue makes out on the horizon to southward another heavy smoke. Weather is fine, wind east, moderate sea.

Required.—Blue commander's estimate of the situation.

Assumptions.—1. BBs have ¾ life and 80 per cent gun power of AAs. At 6000 yards range or less, BBs and AAs have equal life.

2. AAs have 21 knots squadron speed, maximum individual speed 23 knots. BBs are known to have from 23 to 25 knots speed.

3. AAs have two torpedo tubes, one reload, 30-knot 6,000-yard torpedoes.

4. One hour required to get radio message through to Red base.

NOTE: AA=Dreadnought.
BB=Battle cruiser.
V=Destroyer.

ESTIMATE OF THE SITUATION.

Mission.—To capture or destroy the Red convoy.

Enemy Forces.—Their strength, disposition and probable intentions.

The enemy escort of 5 BBs is about equal to our force in gun power but is inferior in protection. Normally its speed is superior, but as the escort and convoy have been proceeding in company at slow speed we have good reason to believe that for a time at least our speed will be superior. The escort is now concentrated and (apparently) about 10 miles distant from the convoy. The enemy commander may decide to call the convoy near the escort, or he may

disperse the convoy and move with the escort to meet us, hoping to decide the issue before we can reach the convoy with any of our force. This latter course would seem to be the more attractive to him, for he may hope, even if defeated, to be able to cripple our ships and thus delay their attack on the convoy till reenforcements can reach him from his base 100 miles away. If he remains concentrated, we have a good chance to get at least one ship around his defence; so he will probably divide his force if we divide ours for a dispersed attack.

Our Own Forces.—Their strength, disposition and courses open to us.

Our own forces are strong enough to have a good chance of success in engaging the enemy escort, but the chance of having our speed reduced by a severe action, and the delay in getting at the enemy convoy which would result from this, may prevent the success of our mission, which is *to destroy the convoy.* Our objective being the convoy, how can we best move to bring destructive force against it? We must take advantage of our presumed temporary superiority in speed, which will continue less than an hour, and of the limited time we are free to act before enemy reenforcements can reach him, keeping in mind the fact that even a single unit of our force, brought into contact with the convoy, can do much, if not all, that is necessary, to accomplish our mission.

There are two general courses open to us:

1. To hold our force together and direct it against the convoy, accepting engagement with the escort only if forced to do so, as we almost certainly will be.

The disadvantage of this course is that we will in all probability find the enemy force thrown directly across our path, while the vessels of the convoy scatter on courses which carry them away from the escort and from us. In the engagement which must ensue, we have a fair chance of winning, but at a price in loss of time and in reduction of speed which will leave us little hope of seriously damaging the convoy.

2. To divide our force and endeavor to get one or more ships around the flank of the escort and in touch with the convoy before the convoy can scatter so widely as to make pursuit ineffective against any considerable number of its units.

This plan has the disadvantage that, in dividing our force, we run the risk of having it crushed in detail. This risk is the more serious because the enemy ships will, within an hour, have a small superiority of speed, and because, moreover, they can expect assistance from the base within from four to six hours. It offers, however, the best possible chance of reaching the convoy before it is widely scattered. And while it is true that the enemy may concentrate against one part of our force, he can do this only by neglecting the other part, thus presenting the opportunity which we seek, and for which we are ready to pay the price—the opportunity of reaching the convoy in time to capture or destroy it.

Decision.—To make a dispersed attack on convoy at maximum individual ship speed.

Maj. John Rodgers Meigs, Corps of Engineers, U. S. Army.[1]

Bvt. Maj. John Rodgers Meigs, eldest son of Gen. Montgomery Cunningham Meigs and Louisa Rodgers Meigs, was born in Washington, D. C., February 9, 1842, and was graduated at West Point Military Academy, July, 1863, No. 1 in his class. He was commissioned July 13, 1863, First Lieutenant, Corps of Engineers, U. S. Army, and immediately sent into the field.

While as a Cadet on leave he was visiting his family in Washington, the Battle of Bull Run took place, and Captain, later, Gen. H. J. Hunt, then commanding a battery of light artillery, made him an Aide on his Staff. As our Army began its retreat, Meigs, seeing a regiment breaking ranks, rode up and told the men to retain their positions and to face the enemy. One of its officers asked: "By what authority are you ordering our men?" he replied: "By the authority of necessity, Sir, for it seems there is no one else to order them!" and this, when he was but 19 years old!

On August 1, 1864, he became the Engineer Officer on the Staff of General Sheridan, then commanding our forces in Shenandoah Valley. He was made Brevet Captain September 19, 1864, for gallant and meritorious services at the Battle of Opequon, Va., and Brevet Major September 22, 1864, for gallant and meritorious services at the Battle of Fishers Hill, Va. He was killed by guerrillas, October 3, 1864, when but 22 years of age.[2]

He was buried in Oak Hill Cemetery, Georgetown. President Lincoln, Secretary of State Seward, Secretary of War Stanton, General Halleck, and other distinguished persons attended his funeral out of respect to his father and his own memory. Later, his body was removed by his father to Arlington Cemetery, where

[1]Portrait (see frontispiece) and sketch of life of John Rodgers Meigs contributed by his uncle, S. Emlen Meigs, Captain and Assistant Quartermaster, U. S. Volunteers, 1861 to 1863.

[2]See "Hist. Rec. U. S. Army," Vol. I, page 1702, by F. B. Heitman.

it now lies beside that of his father, mother, and others of his family.

His name is recorded, along with those of others, on the beautiful marble column at West Point Military Academy, erected by fellow graduates in memory of the graduates who fell during the four years war for the Union and the Constitution.

The following extracts from the sources indicated relate to the services and death of this officer:

From *Official Gazette,* Washington, October 7, 1864—

A despatch from . . . reports an officer of General Sheridan's staff just arrived. General Sheridan was still at Harrisonburg. His supply trains were going on all right, occasionally interrupted by guerrilla parties, the only force on the road. This officer brought in the remains of Lieut. John Rodgers Meigs, of the Engineer Corps, and son of Brevet Major General Meigs, Quartermaster General, who was killed by bushwhackers on Monday last, while making a military survey. In the death of this gallant officer the Department has occasion to deplore no ordinary loss. Last year he graduated at the Military Academy, West Point, with the highest honors, at the head of his class, was commissioned as a Lieutenant of Engineers, and immediately sent into the field. He performed meritorious and dangerous services during the last year on the fortifications at Baltimore, at Harpers Ferry, and at Cumberland, and was made Chief Engineer in the Army of the Shenandoah. In the campaigns he accompanied the army under Sigel, Hunter, and Sheridan. In every position he gave proof of great professional skill, personal courage, and devoted patriotism. One of the youngest and brightest ornaments of the military profession, he has fallen an early victim to murderous rebel warfare. . . .

[Signed] EDWIN M. STANTON,
Secretary of War.

From "Personal Memoirs of P. H. Sheridan," page 467, Vol. I, August, 1864:

At Harpers Ferry . . . sent for Lieut. John R. Meigs, the chief engineer officer of the command, to study with him the maps of my geographical division. . . . I found that, with the aid of Meigs, who was most intelligent in his profession, the region in which I was to operate would soon be well fixed in my mind. Meigs was familiar with every important road and stream, and with all points worthy of note west of the Blue Ridge, and was particularly well equipped with knowledge regarding the Shenandoah Valley. . . .

Ibid., page 50, Vol. II:

. . . About dusk on the evening of October 3, 1864, between

Harrisonburg and Dayton my engineer officer, Lieut. John R. Meigs, was murdered within my lines, . . . The fact that the murder had been committed inside our lines was evidence that the perpetrators of the crime, having their homes in the vicinity, had been clandestinely visiting them, and had been secretly harbored by some of their neighboring residents. Determining to teach a lesson to these abettors of the foul deed—a lesson they would never forget—I ordered all the houses within an area of 5 miles to be burned . . . but when a few houses in the immediate neighborhood of the scene of the murder had been burned, Custer was directed to cease his desolating work, but to fetch away all the able-bodied males as prisoners.

Miscellaneous Notes.

Direct Lift Bridge Across Louisville and Portland Canal.

BY

Maj. J. C. OAKES,
Corps of Engineers; M. Am. Soc. C. E.

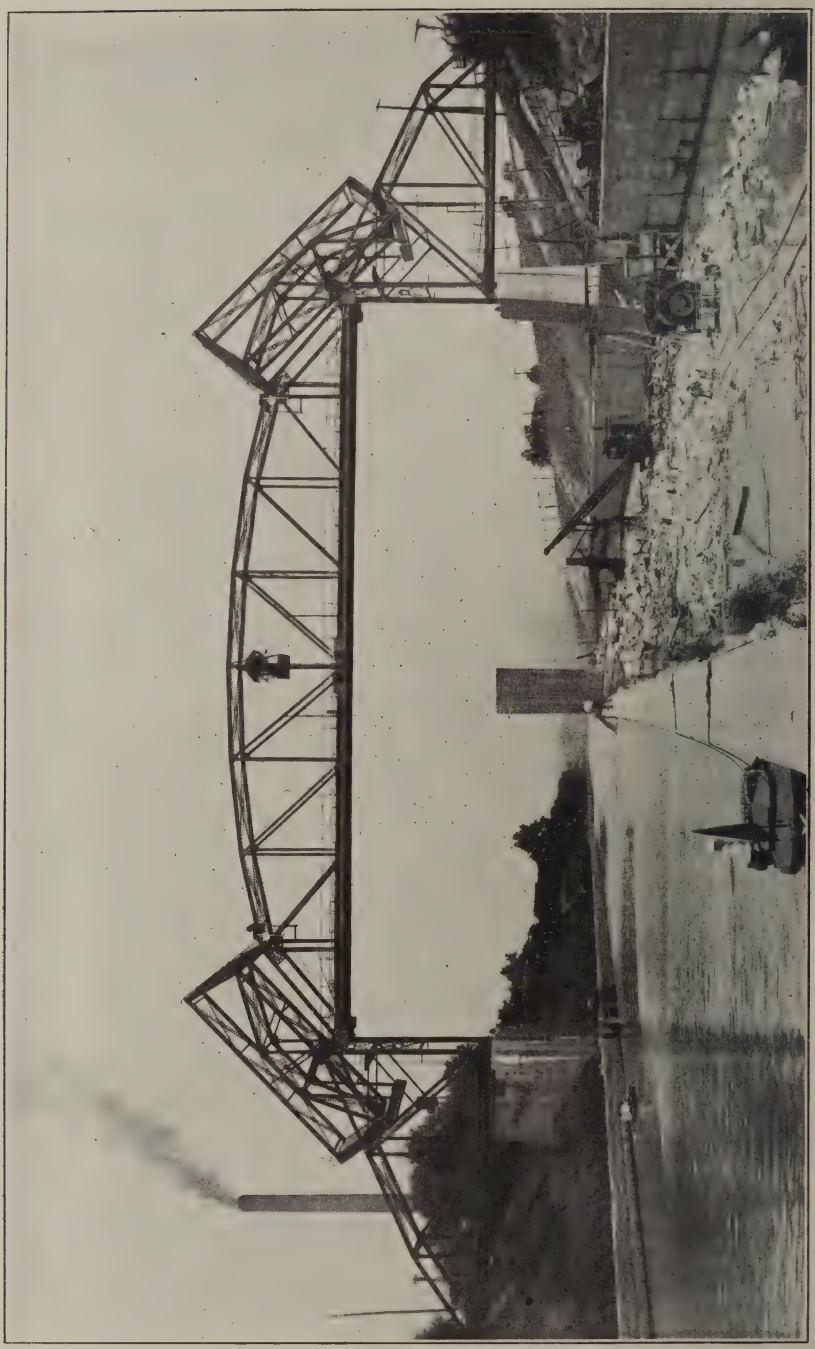
The United States has recently completed a new lift bridge to replace an old swing bridge over the Louisville and Portland Canal at Eighteenth Street, Louisville, Ky. The replacement was made necessary by the widening of the canal from 86½ to 200 feet between vertical walls. The conditions existing at the site and the requirements of navigation made it necessary that a lift bridge should be constructed. After investigation of the usual forms of lift bridges, plans were adopted for the direct lift bridge shown in the accompanying illustrations.

The conditions controlling the design of the bridge were as follows:

Clear width between piers	204 feet, no skew.
Width of roadway	16 feet, no sidewalks.
Lowest point of span when lowered	38 feet above pool.
Lift	40 feet.
Floor, 3-inch oak planks with steel joists.	
Live load for trusses	100 lbs. per sq. ft.
Live load for floor	15 tons on two axles, 10 feet center to center.

The bridge was designed by the Strauss Bascule Bridge Company, acting as consulting engineers, and it was constructed by the Penn Bridge Company, supervised by the Louisville U. S. Engineer Office.

One of the illustrations of this paper shows the lift span in a peculiar position as a result of an accident which occurred on July 16, 1915, a few days after the bridge was completed. A machinist was adjusting the rollers between the lift span and the up-



Direct Lift Bridge, Louisville and Portland Canal, Raised. July 19, 1915.

Direct Lift Bridge, Louisville and Portland Canal, Lowered. July 16, 1915.

Direct Lift Bridge, Louisville and Portland Canal, After Accident. July 16, 1915.

rights. He had removed the pinion that gears into the rack bar on one side at the north end of the bridge, and then removed the key from the pinion on the other side. This action disconnected the operating gear and the rack bars at one end of the span, with the result shown. The span was brought back to its proper position by using a line from a derrick boat in the canal, and a careful examination of the bridge failed to show any injury whatever.

A Season with the Cement Gun.

BY

Maj. W. G. CAPLES,
Corps of Engineers.

Upstream winds and ocean-going steamers produce waves which destroy levees protected only by sod. The wind waves act principally in the heads of reaches, but steamer waves act along the entire course of the vessel. Ocean-going steamers now ply the Mississippi River regularly to above Baton Rouge. In all, some four to five hundred miles of levee will eventually have to be protected against wave action. The high cost and short life of wooden fences are causing their replacement with concrete revetments as rapidly as funds permit. The cost of these revetments is very high. Since levees are built with machines, the cost of the revetment generally about equals and sometimes exceeds the cost of the levee.

In his experiments at Fairfield Levee, Major Sherrill sought, among other things, to reduce the cost of revetment. A layer of gravel was placed on the levee. Into this mortar was blown with a cement gun to form concrete in situ. Penetrations of 4 inches were readily obtained. The resulting revetment, when two inches thick, proved about a strong as a 4-inch layer of 1 : 4 : 7 concrete and cost only two-thirds as much. The experiments were so thorough and promising that the writer purchased a cement-gun plant (Chicago type) and has used it for the past season on levee revetments. The plant has done generally what was expected of it, but in its operation a good many experiences were had, the relation of which may be helpful to other beginners in the use of this machine.

The cement gun was supplied with air through a 2-inch pipe from a 500-foot, straight-line, single-stage compressor and with

a 125-foot receiver set in the line. The consumption of free air at 50 pounds pressure was 300 cubic feet per minute. Five cubic yards of dry materials were handled per hour. The output was limited by difficulties met in field operation. Water was supplied from the river through a ¾-inch pipe.

The principal trouble encountered was clogging of the machine, which was found to be due to three causes—large particles, wet sand, and wet air. The gun is not intended to handle particles retained on a half-inch screen. Larger particles clog the feed wheel and cause about fifteen minutes delay for their removal. The best

Operating the Cement Gun from a Barge—Star Levee, 1915.

sand obtainable would occasionally have some large particles. In the moist atmosphere of this locality, cement stored for any length of time would become lumpy. Wet sand forms lumps with the cement before the mixture can pass through the gun. Uniformity of product requires that the sand and cement be mixed before placing in the gun. All these conditions were met by mixing the sand and cement, screening the mixture through a ½-inch screen, and then feeding the mixture to the gun.

Wet sand causes the cement to set in the gun, destroys gaskets, clogs the feed wheel and delivery hose, and is the most prolific source of trouble. On building construction and coating of metal

structures it is possible to protect the sand supply and even to use sand driers, but on an exposed paving job such measures are impracticable. The best remedy found was to keep the sand in small conical piles that would drain easily. A single day's supply can be protected from the weather, but protracted rains cause a lay-up of the plant and add materially to the expense of the work.

Wet air was the result of a climate having 60 inches of annual

Applying Gunite to Earth Surface—Star Levee, 1915.

The toe-wall, 6 inches by 3 feet, is laid using a mixer. It is not connected rigidly with the face coating, as settlement of the levee breaks any such connection.

rainfall, well distributed, and almost daily fogs during a large share of the working season. The effect of wet air is to cause the cement to set in the air passages, partially closing them and making their cleaning difficult.

The successful manufacture of concrete in situ during the Fair-

field experiments was found to be due to having accidentally obtained a gravel with just the right amount of voids. A gravel containing a large share of small particles can be penetrated only about three-quarters of an inch. Experiments with graded aggregate indicate that the gravel should be all retained on a ¾-inch screen and all passed by a 2-inch screen if concrete is to be made in situ. The local price of such gravel made its use out of the question. The small saving in mortar given by a ¾-inch penetration would not pay for handling the gravel. Thus it became economically impossible to make concrete in situ, and work was confined during the later part of the season to placing a straight 2-inch gunite coating on the levee.

As mortar only was used for the coating, it became necessary to determine the economical mixture. When gunite is applied, from 10 to 25 per cent of the sand rebounds, depending upon nozzle velocity and angle of impact, the sand loss being least at normal impact. The cement all remains, giving a somewhat richer mixture in place than is fed to the gun. Experiments indicated that a 1 : 4 mortar fed to the gun would give a gunite of the proper strength. Briquettes were taken of this gunite blown into the molds and allowed to cure under water. Tensile tests on these briquettes give the following results:

Age (days).	Number of briquettes	Pounds per square inch.		
		Maximum.	Minimum.	Average.
7	3	194	133	162
28	8	394	321	359

The sand used affects the working of the gun. Very fine sand causes the gun to clog. Coarse sand having many particles just passing a ½-inch screen gives the best results. The coarser grains keep the machine and passages scoured clean.

The proper width of panel had to be determined by experiment. The first work was laid in alternate 10-foot panels with elastite joints, each 50 feet. The frequent moves of the gun were expensive and the joints between alternate panels were not satisfactory. The second work was laid continuously with elastite joints spaced 50 feet. Expansion cracks developed generally at 15 to 17 feet each side of each joint, although some panels showed expansion cracks only in the middle. Wire reinforced work showed no advantage over unreinforced. The last work was done by setting elastite

joints each 20 feet and blowing on the gunite continuously. The joints were floated to remove gunite extending across them. The rest of the surface was left as laid without floating. This method appears to be satisfactory.

In addition to the foregoing, certain other information was obtained, to-wit:

A 1:4 gunite is denser and stronger than hand-mixed mortar of the same or considerably richer mixture. For levee paving, reinforcement in gunite is uneconomical and unnecessary if care is taken in placing expansion joints.

Making Concrete in Situ.

The cone is required to keep the wind from blowing away the cement. Goggles are required because of the sand and gravel that rebound from the blast of the gun. The large hose delivers air, sand and cement; the small one delivers water. Hydration takes place at the nozzle.

Expansion joints should be set in place with wire nails, as wooden pegs swell and cause cracking. The replacement of 20 per cent of the cement with hydrated lime reduces cracking, but is difficult to accomplish if the mixing is exposed to high winds.

Gunite bonds almost perfectly with old work. Gunite can be successfully shot through 200 feet of flexible pipe of uniform diameter if the materials are dry and 70 pounds air pressure is used. The delivery pipe will clog at any point where its diameter changes.

The ability to blow the gunite 200 feet explains why a pure mortar coat can be used instead of concrete. The batture is generally from 75 to 200 feet wide. Between the water's edge and the levee is generally a fresh pit full of water. All material has to be moved by hand under exceptionally difficult conditions. Two dollars worth of materials become seven dollars worth of revetment largely through excessive cost of handling. The cost of handling the gunite is much less.

For repair work, the cement gun easily is superior to any mixer, because the gun can be kept on a barge, reducing installation and moving costs to a minimum, and the gunite has a bond with old work superior to any obtainable with ordinary concrete.

Since the work has been in place, too little time has elapsed to tell what will be the ultimate result. Thus far, the 2-inch gunite coating has answered every purpose of a 4-inch concrete coating and has cost only two-thirds as much.

The margin furnished by the cost of handling concrete materials is so great that it seems probable that a concrete plant will be designed specially for levee work and able to do work cheaper than is done by the cement gun. The latter is, however, so excellently fitted for repair work and small jobs that it will have probably no competitor for many years to come.

Fieldwork has been under the charge of Messrs. T. M. McCarroll and E. W. Bres, with Mr. W. E. Knobloch in general charge. Valuable assistance has been furnished by the General Cement Gun Company who, of their own initiative and at their own expense, kept a skilled operator on the ground until the crew became experienced in the use of the gun.

Cross Sections of the Yellow River, China.

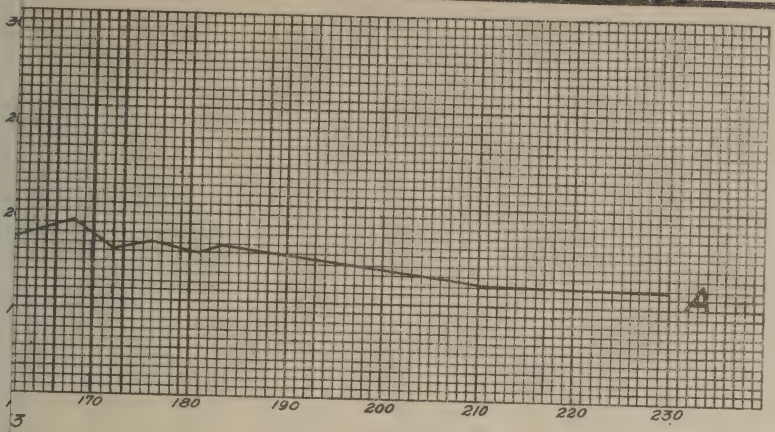
In the discussion of the article on "The Huai River Conservancy Project," printed in PROFESSIONAL MEMOIRS No. 33, it was stated that copies of sections taken of the Yellow River, China, near Kaifengfu would be obtained from China and printed. They are shown on the plate opposite page 656 of this number. Sections A and B are the sections referred to and show the relative elevations of the present river bed and the surrounding country. Section C on the

same plate shows the relative elevation of the river bed in 1853, near the point where it left its old channel.

Errata, No. 33.

Page 268, line 34, for "*cubic meter*" read "*cubic centimeter;*" page 268, line 37, for "*meter*" read "*centimeter.*"

These amendments are made with the sanction of the author of the article who, through recent correspondence with the Suez Canal authorities, has established the fact that, through a misunderstanding, incorrect data were incorporated in the original paper.



ELEVATION IN METERS

ROSS-SECTIONS OF YELLOW RIVER, CHINA.

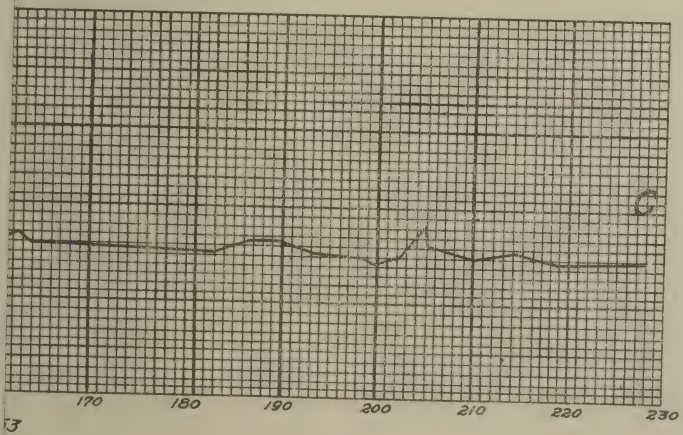
Supplementary to the Article entitled

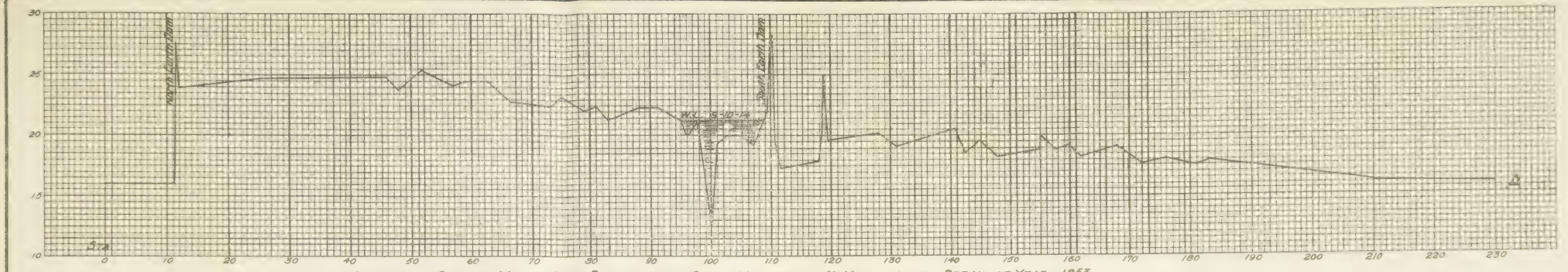
"The Rise of the Bed of the Yellow River by the Deposit of Sediment"

Printed in Professional Memoirs, No. 33, pp. 395 to 400, and referred to on page 400 of that article.

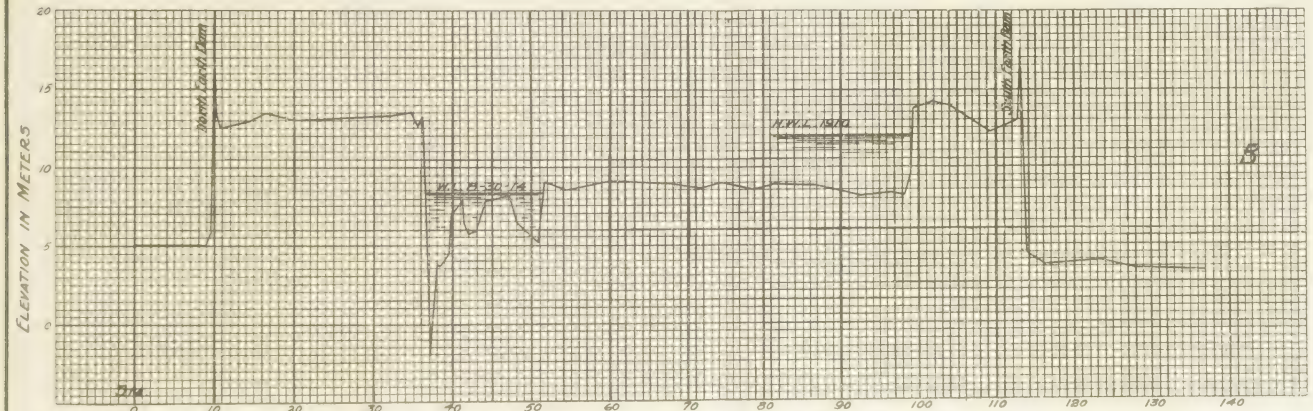
Note:

Cross-section "C" shows the relative elevation of the river bed in 1853 near the point where it left its old channel.





SECTION OF PRESENT YELLOW RIVER BED OPPOSITE CHUNG MOW ABOUT 40 MILES ABOVE BREAK OF YEAR 1853



SECTION OF PRESENT YELLOW RIVER BED OPPOSITE KAO CHUANG ABOUT 7 MILES ABOVE BREAK OF YEAR 1853

CROSS-SECTIONS OF YELLOW RIVER, CHINA.

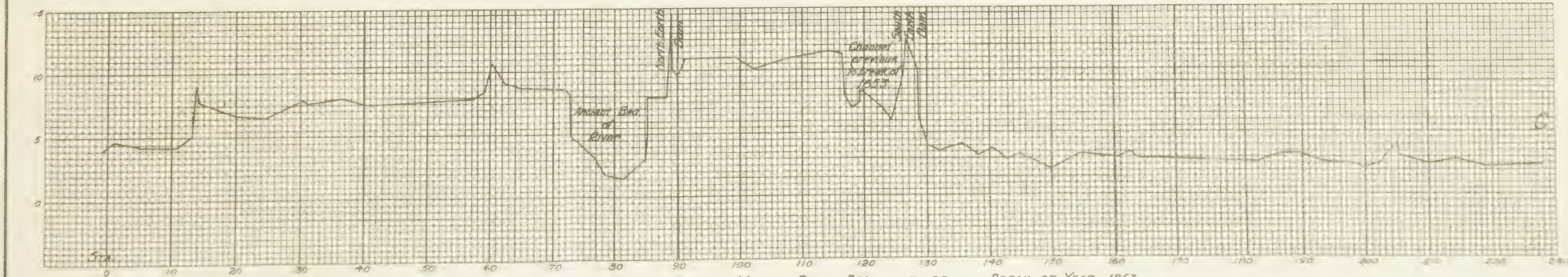
Supplementary to the Article entitled

"The Rise of the Bed of the Yellow River by the Deposit of Sediment"

Printed in Professional Memoirs, No. 33, pp. 395 to 400, and referred to on page 400 of that article.

Note:

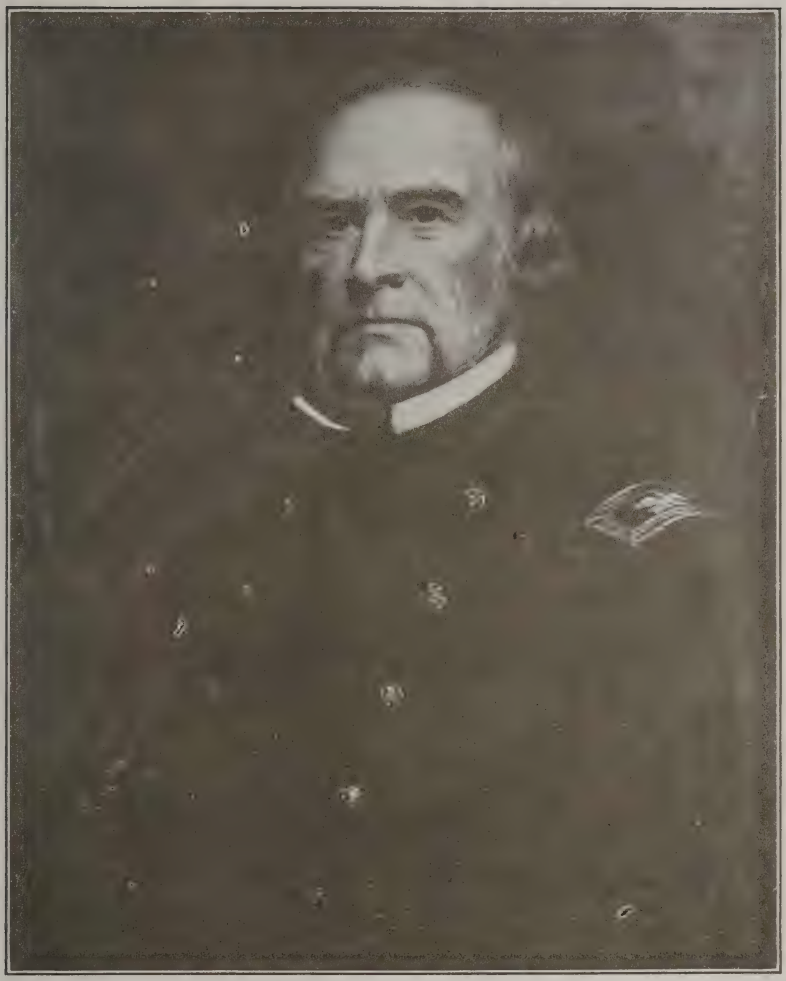
Cross-section "C" shows the relative elevation of the river bed in 1853 near the point where it left its old channel.



SECTION OF OLD YELLOW RIVER BED JUST BELOW BREAK OF YEAR 1853

ELEVATION IN METERS

DISTANCE IN HECTOMETERS



RENÉ EDWARD DE RUSSY

BORN 1789—DIED 1865.

Cadet, 1807-1812
Second Lieutenant, Corps of Engineers, 1812
First Lieutenant, Corps of Engineers, 1812
Brevet Captain, U. S. Army, 1814
Captain, Corps of Engineers, 1815
Brevet Major, U. S. Army, 1824
Major, Corps of Engineers, 1830
Brevet Lieutenant Colonel, U. S. Army, 1834
Lieutenant Colonel, Corps of Engineers, 1838
Colonel, Corps of Engineers, 1863
Brevet Brigadier General, U. S. Army, 1865

SEE PAGE 758

PROFESSIONAL MEMOIRS

Corps of Engineers, United States Army, and Engineer Department at Large

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Hydrology of the Isthmus of Panama.

BY

Brig. Gen. HENRY L. ABBOT,
United States Army, Retired.

The Panama Canal being now opened to traffic, there remains for study only one important hydraulic problem—the sufficiency of the available water supply to meet the needs for lockage, for mechanical power to operate the canal and railroad, and for the electric lighting of the Canal Zone. It should not be forgotten, however, that if a larger volume of water be desirable for these and other uses, the plan proposed by the New French Company to supplement the volume of its smaller projected lake is still available; namely, the construction of a masonry dam near Alhajuela, where a good site exists for creating an upper lake to hold back the surplus water which during the rainy months now runs to waste through the spillway.

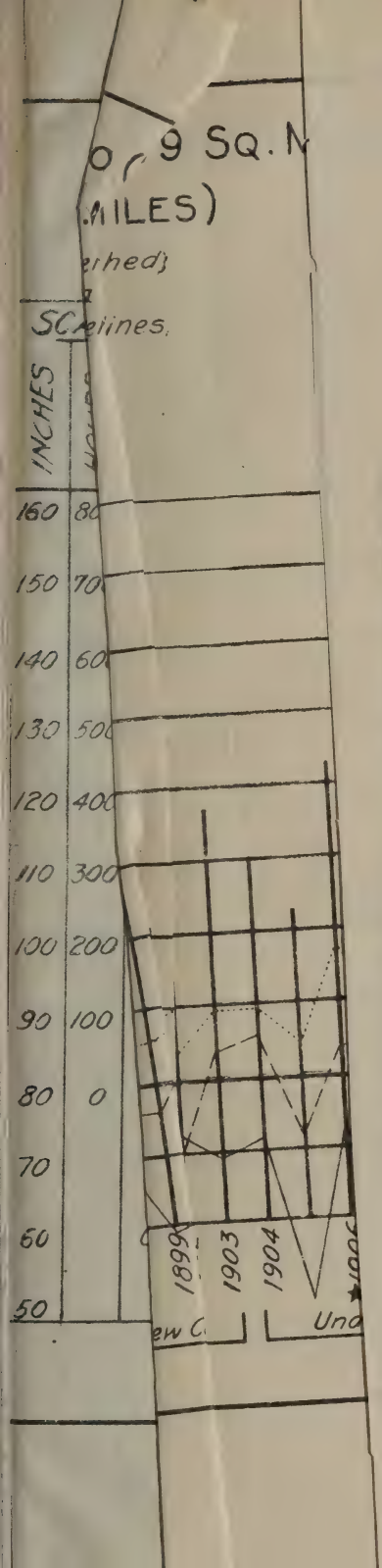
The Canal Zone lies between the ninth and tenth degrees of north latitude in a region of exceptional rainfall; where the sun, closely followed by rain clouds raised from the oceans in his annual journey between the tropics, exerts a controlling influence upon the volume of local rainfall. When near his southern limit, in January, February, March and April, precipitation upon the Isthmus is at its minimum; December and May are usually intermediate in volume; in the remaining six months, when near his northern limit, heavy downpours are the rule. Furthermore, the local annual rainfall is not uniform across the Isthmus. As one passes from the Atlantic to the Pacific coast, the volume falls off gradually from about 130 inches at Colon to about 70 inches at Panama. It will be noticed that even the latter is more than double the usual downfall in the United States, a fortunate circumstance for our great artery of commerce. Another local advantage is the fact that the atmosphere of the Isthmus is nearly saturated with aqueous vapor, which largely reduces the losses by lake evaporation. Our hydraulic problem seems therefore to be specially concerned

with Isthmian rainfall and outflow, and the relation between them.

To determine accurately the average annual rainfall at any locality, the records should cover at least half a century. Although the Isthmus has been known to civilization for more than four hundred years, the first annual rainfall records date from 1863, when they were begun by the agents of the Panama Railroad Company. As to the outflow from the watershed, the records are even less complete. The volume received from the clouds, after reduction by evaporation, by plant growth, and by possible infiltration, represents the available flow at the dam site. To determine to what extent existing data throw light upon this quantity, has seemed to me to be of primary interest.

The earlier records are given in a paper published in the *Monthly Weather Review* in May, 1899; they include those of the Railroad Company, and those at that date collected by the two French Companies and the Liquidation. A gap of a few years in the former is supplied by a paper by Mr. A. P. Davis, published in the Twenty-second Annual Report of the United States Geological Survey for 1900-1901. These data are supplemented and discussed by the writer in papers published in the *Monthly Weather Review* of February, 1904, and of June, 1908, together with some later data collected by the American engineers. In March, 1913, Mr. Caleb M. Saville contributed a valuable paper upon the Hydrology of the Canal to the Transactions of the American Society of Civil Engineers, bringing the records up to 1910. The Annual Reports of the Isthmian Canal Commission, of course, cover the American operations. The first point for consideration is, how these data can best be grouped for study?

The narrow limits to which the earlier observations were restricted suggest that the basin of the Chagres River above Bohio should be adopted until Gatun Lake began to fill early in 1910. Its area has been accurately determined by recent surveys to be 779 square miles. The early rain records were mostly restricted to Colon and Gamboa. Fortunately, Colon is situated near the Atlantic Coast line where the rainfall is largest, and Gamboa well represents the Pacific limit of the watershed. A careful analysis of the more ample records of recent years (1898-1907) has shown that the average rainfall in the basin above Bohio is about 89 per cent of that at Colon, 124 per cent of that at Gamboa, and 52 per cent of the aggregate of the two stations. To avoid a change in the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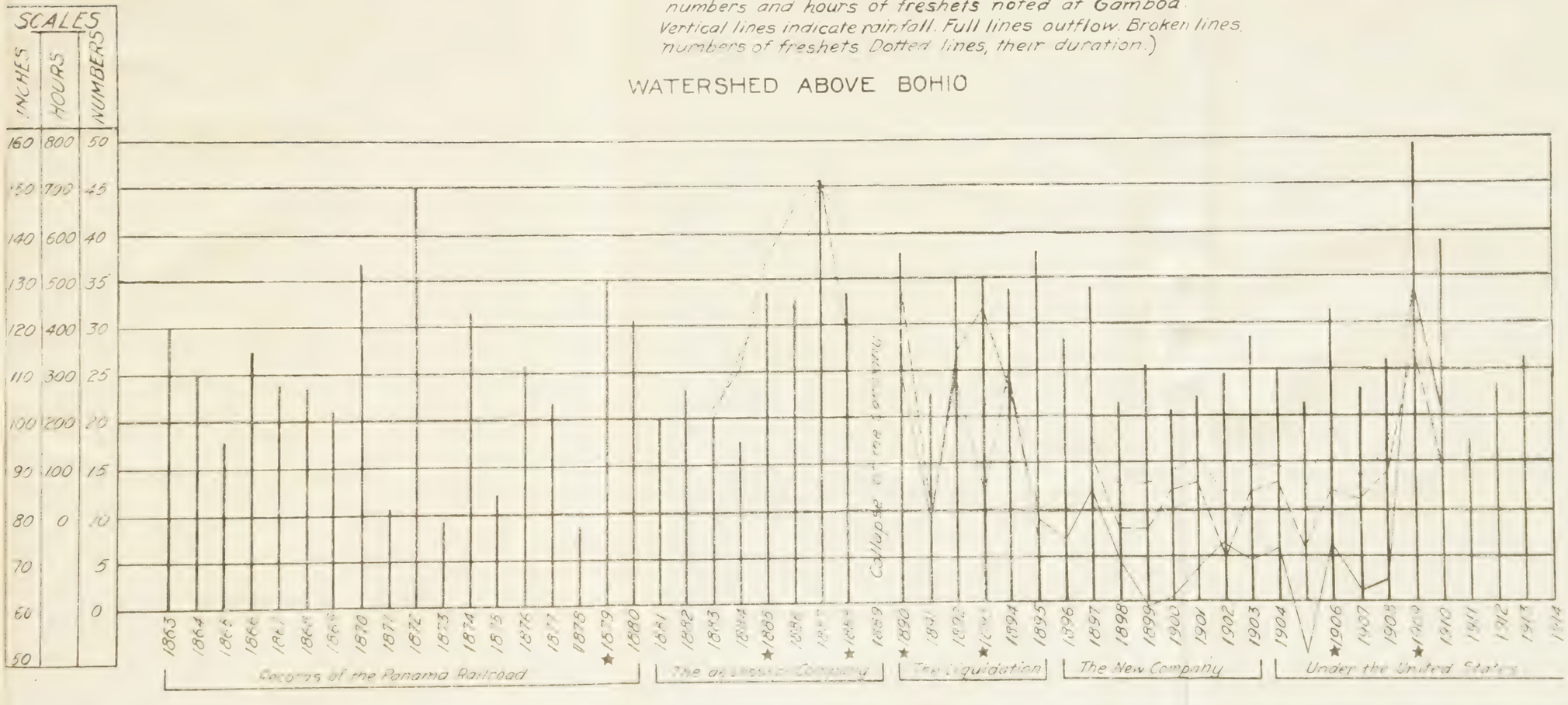
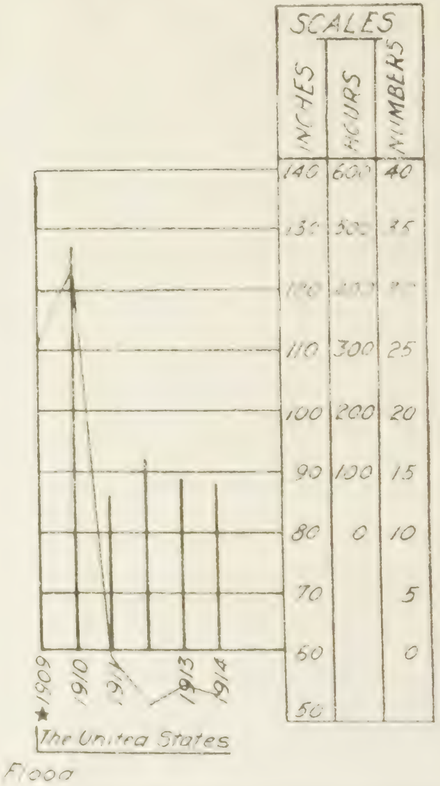
HYDROGRAPHY OF THE WATERSHED ABOVE BOHIO (779 SQ. MILES)
AND OF THAT OF GATUN LAKE (1320 SQ. MILES)

(Annual rainfall and net outflow in inches on watershed; numbers and hours of freshets noted at Gamboa. Vertical lines indicate rainfall. Full lines outflow. Broken lines, numbers of freshets. Dotted lines, their duration.)

WATERSHED ABOVE BOHIO



WATERSHED ABOVE GATUN LAKE



★ Flood

standard, these ratios have been used throughout the following table to estimate the annual rainfall in the basin; and where a few dry months are missing in the early railroad records the vacancies have been supplied by the mean values of the missing months.

The Chagres River is a torrential stream, and the first French Company early established a fluviograph at Gamboa to register continuously the heights of the water. This record has been carefully kept since 1883; until the rise of the lake in 1910 began to affect the local water level. At the times of freshets the oscillations are so sudden that their number and duration are readily noted, and they furnish valuable checks upon the discharge estimates, as may be seen from the Plate. For this study they are assumed to begin and end at a stage 10 feet above low water, which is rarely exceeded.

Accurate measurements of discharge were inaugurated by the Liquidation and the New French Company, the continuous record dating from 1890. The annual outflow is given in the tables under the forms of cubic feet per second and depth in inches upon the watershed; the latter to permit a direct comparison with the rainfall. Years of great floods are also indicated, with the same object in view.

The first table is intended to present, for the basin above Bohio, all available hydraulic data prior to the date when Gatun Lake began to fill, with some later figures added for comparison with those of the second table, which includes the entire watershed above Lake Gatun during the filling of the lake. In future studies this latter basin will doubtless be adopted, as the net outflow is easily determined and rainfall measurements are now made at many stations. For the second table I have adopted the mean of the records at nine stations—Colon, Gatun, Trinidad, Camacho, Empire, Culebra, Gamboa, Alhajuela and El Vigia—as well representing the average volume falling upon the lake watershed near the Canal Zone. Other stations to cover the western part of the lake are desirable, as it will be noticed that the tables indicate a somewhat larger rainfall upon the basin above Bohio than upon that of the entire lake. More western stations would probably correct this. The Plate is intended to present these hydraulic data in a form to be readily appreciated by the eye.

Hydrology of the Watershed above Bohio; 779 square miles.

Year.	Precipitation.		In the basin above Bohio.			Gamboa freshets.		Authority.
	Colon. Inches.	Gamboa. Inches.	Rainfall, inches.	Outflow— Ft.-sec.	Outflow— Inches.	Annual No.	Duration, hours.	
1863	134.3	------	120	------	------	------	------	Panama Railroad Records.
1864	123.4	------	110	------	------	------	------	
1865	107.4	------	96	------	------	------	------	
1866	129.7	------	115	------	------	------	------	
1867	120.8	------	108	------	------	------	------	
1868	120.0	------	107	------	------	------	------	
1869	114.8	------	102	------	------	------	------	
1870	149.6	------	133	------	------	------	------	
1871	99.6	------	81	------	------	------	------	
1872	168.5	------	150	------	------	------	------	
1873	87.1	------	78	------	------	------	------	
1874	137.7	------	123	------	------	------	------	
1875	94.7	------	84	------	------	------	------	
1876	124.5	------	111	------	------	------	------	
1877	115.5	------	103	------	------	------	------	
1878	86.7	------	77	------	------	------	------	
*1879	164.2	------	130	------	------	------	------	
1880	136.4	------	121	------	------	------	------	
1881	102.3	89.5	100	------	------	------	------	de Lesseps
1882	124.1	79.6	106	------	------	------	------	French Cie
1883	115.3	76.6	100	------	------	21	201	
1884	86.5	95.9	95	------	------	25	334	
*1885	146.3	97.5	127	------	------	37	437	
1886	137.2	102.9	125	------	------	43	437	
1887	154.9	136.2	151	------	------	46	709	
*1888	------	102.6	127	------	------	29	596	
1889	------	75.7	----	------	------	------	----	Collapse
*1890	154.3	105.0	135	6,304	110.0	34	308	Liquid'n.
1891	124.7	77.7	105	4,476	78.1	10	115	
1892	145.3	104.4	130	6,513	113.6	24	269	
*1893	131.9	117.8	130	7,081	123.3	12	220	
1894	153.7	90.6	127	6,098	106.4	25	263	
1895	151.5	------	135	4,482	78.2	7	54	The New
1896	131.5	------	117	4,216	73.5	------	----	French Cie
1897	138.0	107.8	128	4,830	84.2	16	157	
1898	115.5	82.6	103	3,944	68.8	8	51	
1899	133.0	80.0	111	3,384	59.0	8	57	
1900	116.1	78.7	101	3,509	61.2	12	84	
1901	107.7	91.6	104	3,855	67.2	13	107	
1902	112.6	97.7	109	4,179	72.9	5	37	
1903	126.3	99.3	117	3,958	69.0	12	94	
1904	126.9	84.5	110	4,110	71.7	13	94	U. S. in
1905	115.4	82.2	103	2,800	48.8	6	48	Charge.
*1906	138.1	97.8	123	4,169	72.7	12	178	
1907	125.6	78.1	106	3,597	62.7	11	69	
1908	137.7	77.5	112	3,732	65.1	14	131	
*1909	183.4	122.1	159	7,335	128.0	27	466	
1910	150.0	116.0	138	6,340	110.6	15	77	Lake began
1911	112.8	70.6	95	------	------	------	------	to fill.
1912	117.6	89.1	107	------	------	------	------	
1913	131.2	86.3	113	------	------	------	------	
1914	132.7	77.4	109	------	------	------	------	

*Great flood of the Chagres River.

Hydrology of the Watershed above Lake Gatun; 1,320 square miles.

Year.	Lake above mean sea level feet.	In the Lake watershed.			Lake evaporation. Foot-seconds.
		Rainfall, inches.	Net outflow—		
			Foot-seconds.	Inches.	
*1909	3.63	137.0	10,704	110.1	----------
1910	13.07	127.2	11,938	122.9	----------
1911	15.15	85.9	5,710	58.8	----------
1912	31.24	92.3	4,985	51.3	----------
1913	57.87	88.9	5,272	54.3	484
1914	85.26	88.2	5,118	52.7	704

*Great flood of the Chagres River.

An inspection of the Plate illustrating these tables shows a satisfactory correspondence between the observed annual rainfall and the outflow and river oscillations as indicated by the number and duration of the freshets. There is, furthermore, a suggestion of a tendency to a progressive annual change between years of maxima and minima in these quantities, which is worthy of notice. If future records confirm this suggestion, it will be needful to study the reason for it, as is now the case with solar spots, aurora borealis, and other natural phenomena. As to the second table, it must not be forgotten that it covers a period when the area of the lake was gradually increasing, causing a variable loss due to evaporation. The values given for it for the last two years are taken from the official reports, and are based probably on direct measurements.

The tropical conditions of the Canal Zone are so different from those in the Continental United States that it is interesting to compare them. This may be done from Rafter's tables, in his valuable paper published as the Water-Supply and Irrigation Paper, No. 80, of the Geological Survey. It is done in the following table.

Average Rainfall, Run-off, and Difference.

Watershed.	Area, square miles.	Period years.	Rainfall, inches.	Outflow losses (evap., etc.):			
				Inches.	Per cent	Inches	Per ct.
Muskingum River	5,828	8	39.7	13.1	33	26.6	67
Genessee River	1,070	9	40.3	14.2	35	26.1	65
Croton River	339	23	49.4	22.8	46	26.6	54
Lake Cochituate	19	38	47.1	20.3	43	26.8	57
Sudbury River	78	26	46.1	22.6	49	23.5	51
Mystic Lake	27	18	44.1	20.0	45	24.1	55
Neshaminy Creek	139	16	47.6	23.1	49	24.5	51
Perkiomen Creek	152	16	48.0	23.6	50	24.4	50
Tohickon Creek	102	15	50.1	28.4	57	21.7	43
Hudson River	4,500	14	44.2	23.3	53	20.9	47
Pequannock River	64	9	46.8	26.8	57	20.0	43
Connecticut River	10,234	8	43.0	22.0	51	21.0	49
Chagres River	779	21	118.1	82.1	69	36.0	31
Gatun Lake Basin	1,320	6	103.3	75.0	73	28.3	27

An inspection of the last four columns will show how much more favorable to the operation of a canal are the hydraulic conditions prevailing upon the Isthmus than those existing near the Atlantic Coast of the United States. Considerably more than double the volume of rainfall is available, and the losses from evaporation, plant growth, and percolation are represented by only about 30 per cent as against 50 per cent of that volume. As stated above, it remains to be determined by the observations of many future years whether there is a tendency to a normal variation in annual rainfall upon the Isthmus, as seems to be suggested by the figures already of record.

Quartermaster and Submarine Mine Wharf at Key West, Fla.

BY

Maj. J. R. SLATTERY,
Corps of Engineers.

Between the shore line at Fort Taylor and the channel leading into Key West Harbor, lies a coral reef with insufficient depths over it for the needs of mine planters. There is but little soft material overlying the reef—entirely too little to hold piles driven only to the rock. The life of untreated wooden piles in these waters is little over a year. It was therefore determined to build a reinforced-concrete pile wharf, and in order to save the expense of blasting holes for the piles, a reinforced-concrete pile designed and patented by Mr. J. M. Braxton, Assistant Engineer, and especially suited for hard driving under conditions where blasting would ordinarily be necessary, was adopted. The general features of this pile are shown on Plate V. In driving, a heavy pipe is placed over the pile, the lower end of the pipe resting on the shoulder of the pile point, and the blows of the hammer are delivered on a suitable driving head on the other end of this pipe instead of on the head of the pile. When a sufficient penetration has been secured in the rock, the pipe is withdrawn and used again.

As originally designed (see Plate V), the piles in each bent were braced by steel diagonal braces, and the caps, stringers and decking were all of reinforced concrete. Longitudinal bracing similar to that shown was also introduced between pile bents. No horizontal system of bracing, however, was used. The piles were driven from 3 to 6 feet into the rock.

The wharf had been just practically completed at a cost of $61,460, when Key West was visited by a hurricane of exceptional severity (October 17, 1910). After withstanding a terrible battering, the approach from the shore to the head of the dock was completely destroyed, the decking on the head of the dock was broken into pieces, and sixteen pile clusters were knocked over.

From information gathered from eye-witnesses, it was thought that the destruction of the wharf was mainly attributable to the reinforced decking which, being rigidly attached to the stringers, afforded no relief whatsoever for the heavy wave action under the decking, tending to lift it bodily, or the pounding of the breaking waves on top of the decking.

When funds became available for reconstruction, it was decided to introduce a system of horizontal bracing in the approach in addition to the bracing originally used, to reinforce caps and stringers at the top as well as in the bottom, so as to better withstand wave action underneath the decking, and to place a plank instead of a reinforced-concrete decking, with the idea that in case of another severe hurricane only the plank decking, which could be renewed at relatively small cost, would be torn off and the wharf be left otherwise intact. The general features of the wharf as reconstructed are shown in Plates I, II and III. The total cost of reconstruction was approximately $41,425.

The following are the detailed field costs of the principal features of the work, both for original construction and for reconstruction:

Original Construction.

Current wages:	*Per Day.*
Laborers	$1.50
Carpenters	3.00
Foremen	4.00

A. Concrete piles:

Total number of feet piling, 10,241.

1. Forms—

Made 15 forms for 30-foot piles—

Materials	$329.26
Labor, making forms, setting up and removing	1,587.98
	$1,917.24
Cost forms per foot piling	.1872

2. Making piles—

Mixing, wheeling and placing concrete	$978.11
Placing steel reinforcement (steel badly rusted and had to be scraped)	536.44
	$1,514.55
Cost per foot piling	.1475

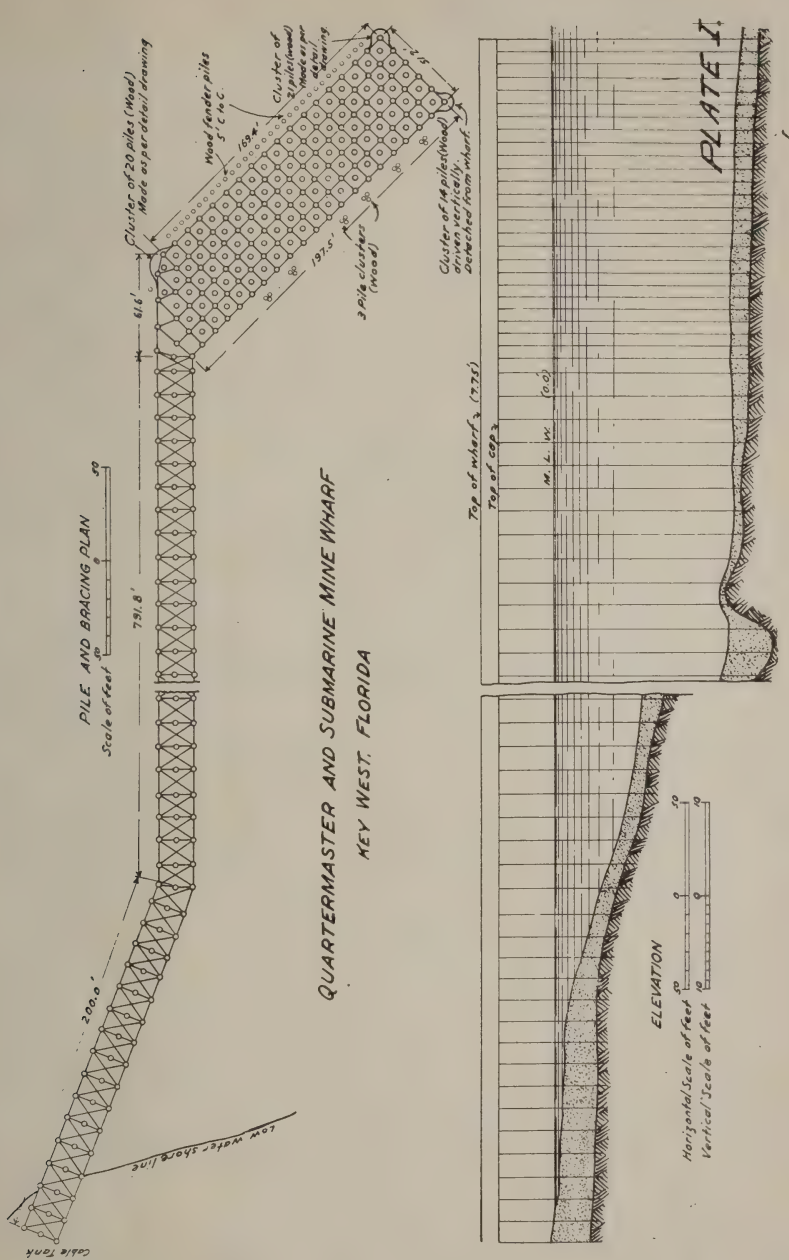


PLATE I.
QUARTERMASTER AND SUBMARINE MINE WHARF
KEY WEST, FLORIDA
PILE AND BRACING PLAN
Scale of Feet
ELEVATION
Horizontal Scale of Feet
Vertical Scale of Feet
Cable Tank
Low Water Shore Line
200.0'
791.8'
61.6'
Cluster of 20 piles (Wood)
Wood fender piles
5' C to C
Cluster of 3 piles (Wood)
3 Pile clusters (Wood)
197.5'
Cluster of 14 piles (Wood) driven vertically, detached from wharf.
Top of wharf
Top of coal
M. L. W.
(0.0)

3. Materials used in making piles—

641 barrels cement at $1.63		$1,044.83
400 pile shoes		1,513.22
8,200 steel hoops		308.43
78,364 pounds steel rods at 2 cents		1,567.28
Royalty on concrete piling		1,024.10
		$5,457.86
Cost per foot piling		.5329

NOTE.—Crushed brick and sand were on hand from other work. If they had been bought, cost would have been:

Stone, 350 cubic yards, at $4	$1,400.00	
Sand, 175 cubic yards, at $2.75	481.25	
		$1,881.25
Cost per foot piling		.1834

4. Total cost of pile per foot—items 1, 2, 3 ..... .8676
If stone and sand had been purchased 1.05

B. Placing, driving, and lining up piles, exclusive of hire of pile-driving plant:

Number of piles driven, 399.

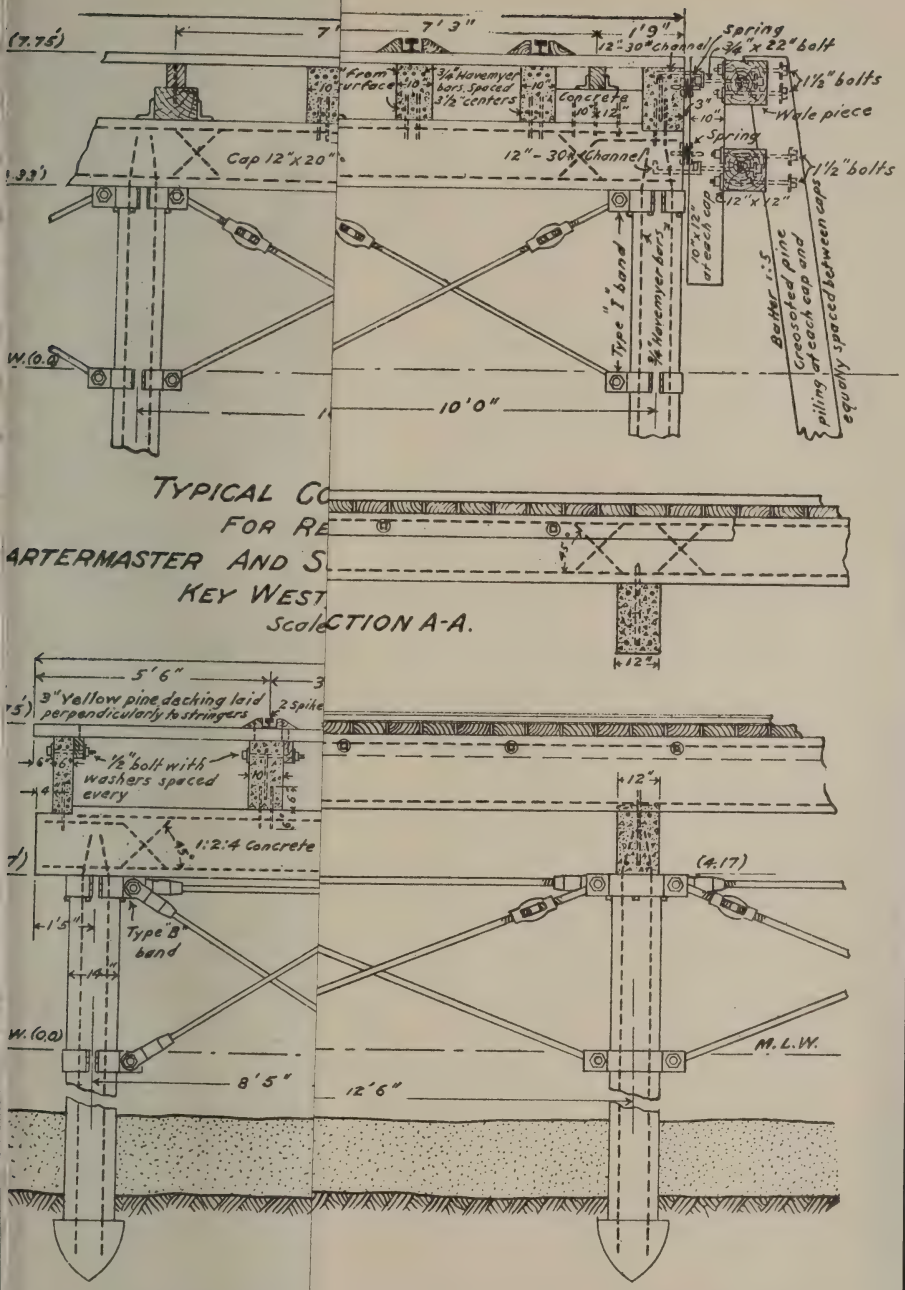
Driving, lining up and cutting off piles	$1,497.44
Hauling and storing piles	276.45
	$1,773.89
Cost per pile	4.45
Cost per foot piling	.17

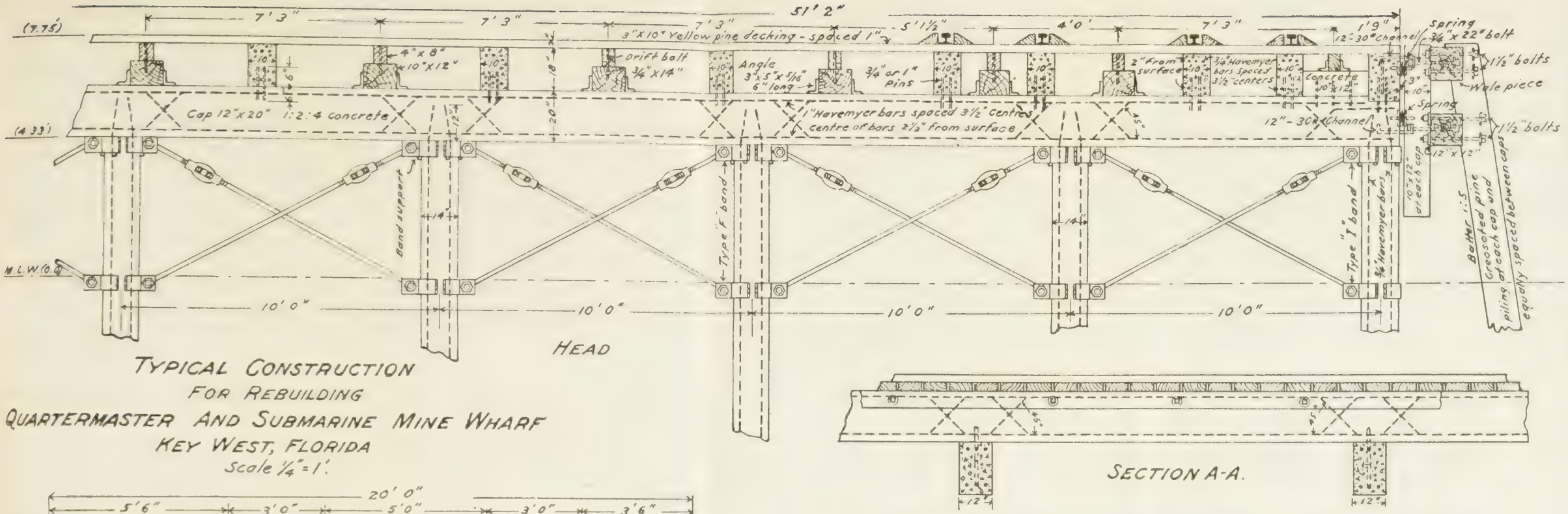
C. Pile-driver plant:

Number of piles driven, 354.

	Per Month.	
1 floating driver on lighter, complete	$200.00	
1 lighter	100.00	
1 ponton	10.00	
1 launch, part time (no charge to this work).		
Hire of plant 5 2-3 months, at $310		$1,757.00
Driving steel shell	$540.00	
Pulling shackles	12.00	
Drive head	76.00	
Coal	132.00	
Rope, oil, waste	340.00	
		1,100.00
		$2,857.00
Cost per pile		8.07
Cost per foot piling		.30

Maximum number of piles driven in one day, 11. Much delay in driving piles caused by rough weather.

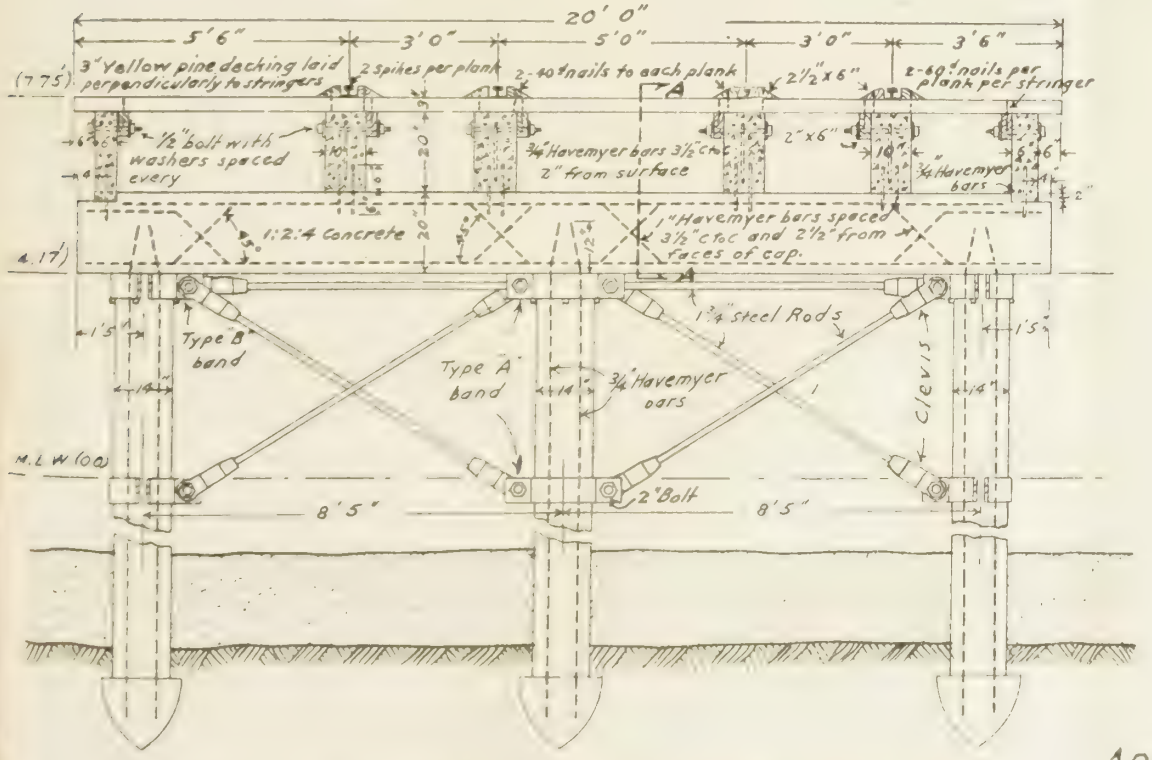

(7.75')
7'
7'3"
1'9"
12"-30" Channel
Spring
¾" x 22" bolt
1½" bolts
Wale piece
Spring
1½" bolts
12" x ½"
10"x12" at each cap
Batter 1:5
Creosoted pine piling at each cap and 4 spaced between caps
Type "I" band
¾" Havemeyer bars
Cap 12"x20"
12"-30" Channel
10'0"
W.(0.0)
Concrete
10"x12"
3"
10"
3¼" Havemeyer bars. Spaces 3½" centers
From surface
10"
10"
10"



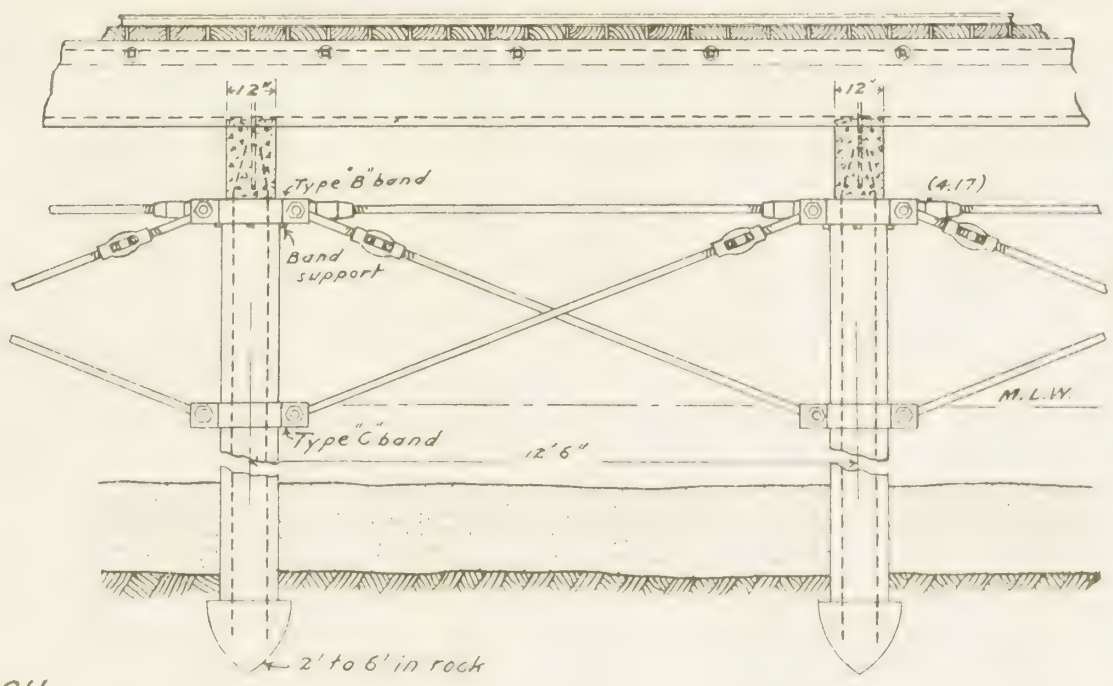
HEAD

TYPICAL CONSTRUCTION
FOR REBUILDING
QUARTERMASTER AND SUBMARINE MINE WHARF
KEY WEST, FLORIDA
Scale ¼"=1'.

SECTION A-A.



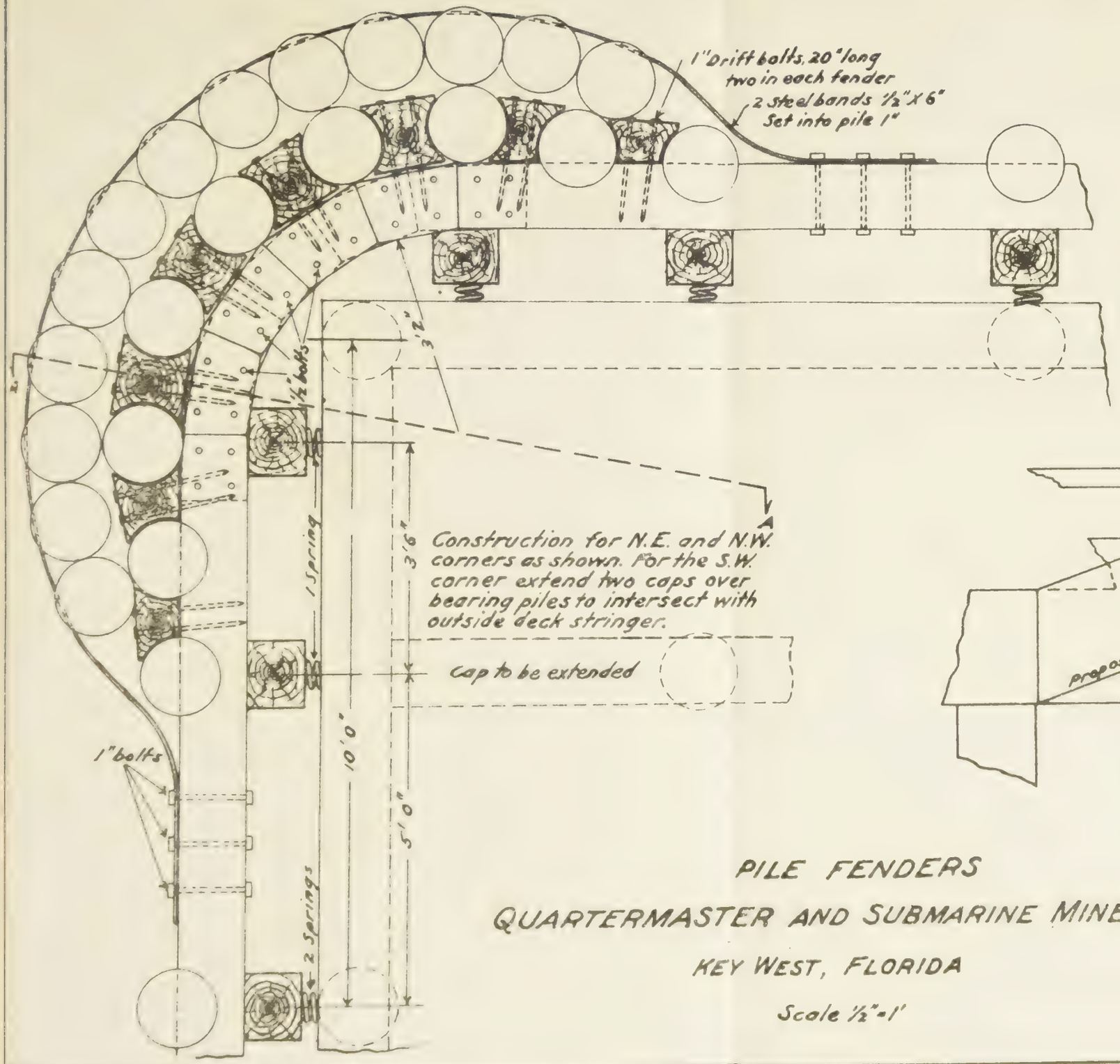
APPROACH

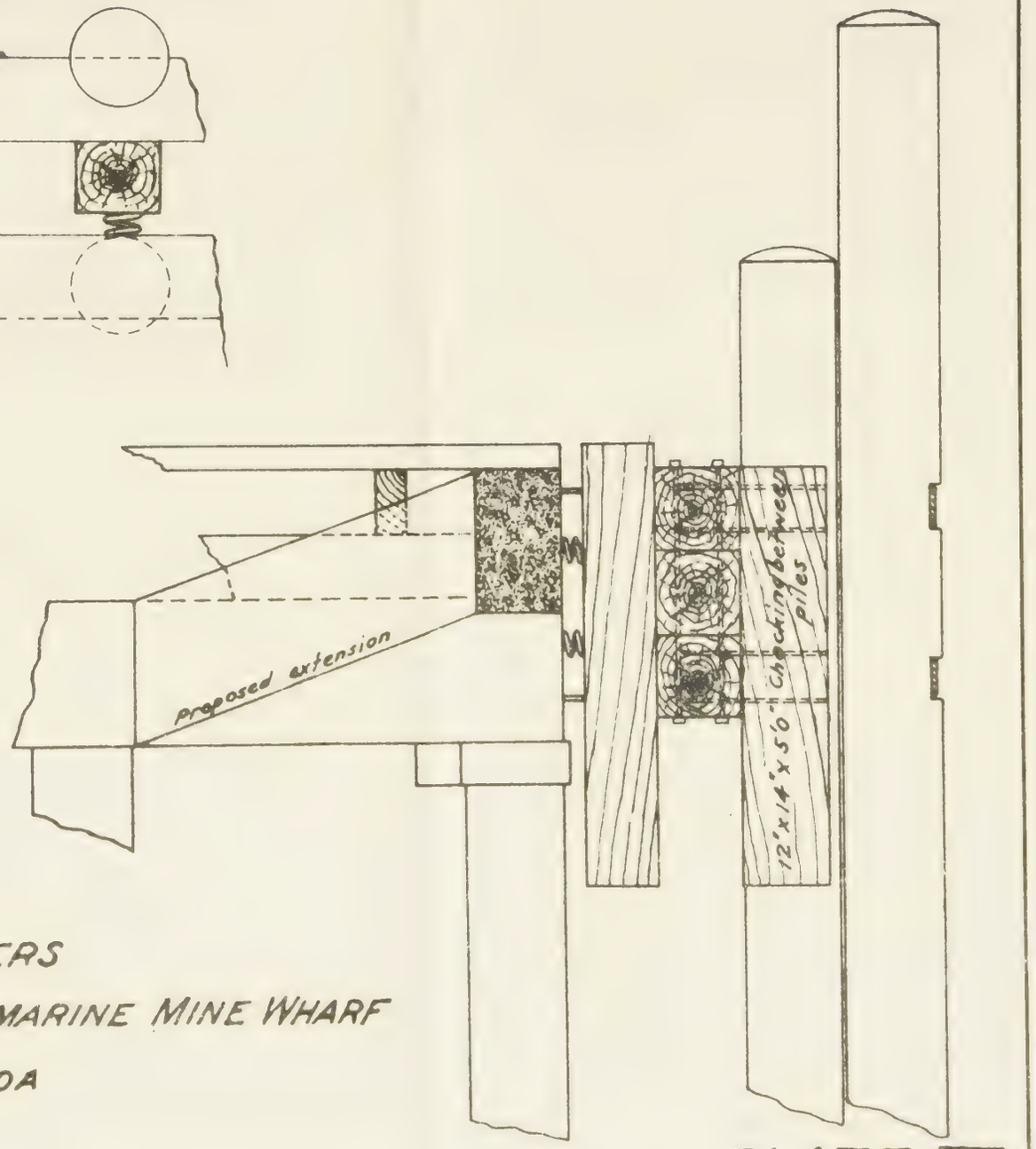


Designed under the direction of J.R. SLATTERY, Capt. Corps of Engineers. By J.M. BRAXTON, Asst. Engineer. August 9, 1912.

PLATE II

PLATE III





PILE FENDERS

QUARTERMASTER AND SUBMARINE MINE WHARF

KEY WEST, FLORIDA

Scale ½"-1'

PLATE III

D. Pile bracing:

Material, 132,500 pounds, at $0.043	$5,700.00
Placing bracing	1,589.00
	$7,289.00
Cost per pound in place	.055

E. Reinforced concrete in caps, stringers and decking:
Total number of cubic yards, 795.

1. Forms—

Materials, lumber, bolts, etc.	$630.33
Making, placing and removing	6,544.00
	$7,174.33
Cost forms per yard concrete	9.03

2. Materials for concrete—

Cement, 1,359 barrels, at $1.64	$2,228.76
Twisted steel, 84,650 pounds, at $0.0169	1,429.21
Electric welded fabric, 19,800 pounds, at $0.0564	1,032.30
Sand, 360 cubic yards, at $2.75	990.00
Value of broken stone, 750 cubic yards, at $4	3,000.00
	$8,680.27
Cost per cubic yard	10.92

3. Preparing concrete—

Mixing, wheeling and placing	$3,291.31
Placing, reinforcement and cleaning	898.59
	$4,189.90
Cost per cubic yard	5.27

4. Total cost per cubic yard, items 1, 2, 3 $25.22

Reconstruction.

Current wages:	*Per Day.*
Laborers	$1.50-$1.75
Carpenters	3.50
Foremen (average)	3.54

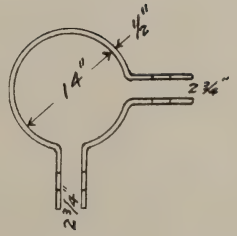
A. Concrete piles:
Total number of piles made, 286.
Total number of feet piling, 6,448.5.

1. Forms—
Made 14 forms for 30-foot piles—

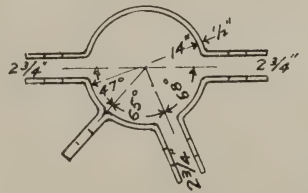
Materials for repairs (forms used from former work)	$96.20
Labor setting up and removing and repairs	1,062.36
	$1,158.56
Cost forms per foot piling	.18

PLATE IV.

Iron and Steel Fittings
Quartermaster and Submarine Mine Wharf
Key West, Florida



Type E



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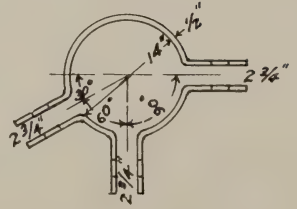

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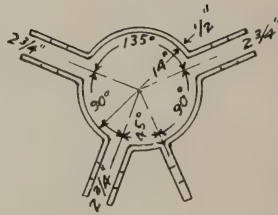

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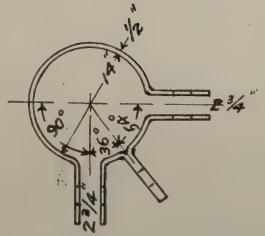

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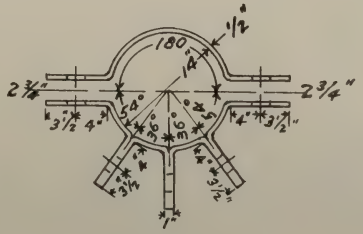
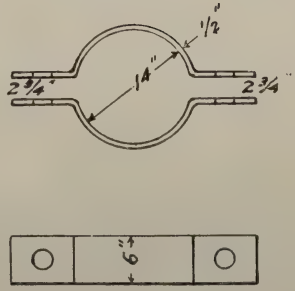
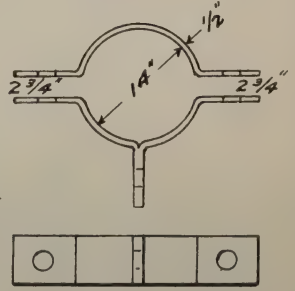


Type B



Type A



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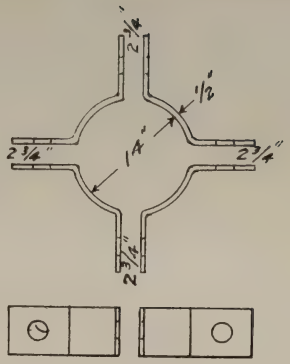

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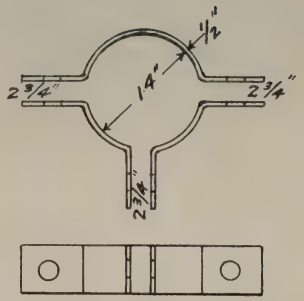
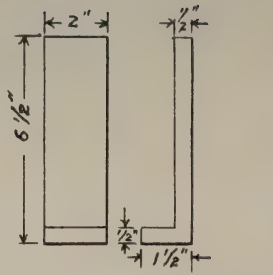
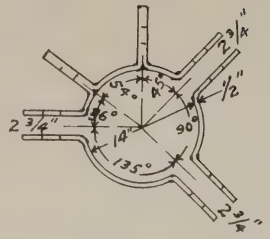


Type I



Steel Clips for Pile Bands
Type R
Scale 2" = 1'



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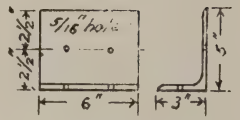


Angle Clips (Steel)
Type W
Scale 1" = 1'

Steel Pile Bands, 6 inches wide, $\frac{1}{2}$-inch thick. Holes for 2-inch bolts. Center of holes 3 inches from edges, $3\frac{1}{2}$ inches from ends. Scale, $\frac{1}{2}$ inch = 1 inch.

2. Making piles—	
Mixing, wheeling and placing concrete	$980.56
Placing steel reinforcement	372.94
	$1,353.50
Cost making piles per foot	.21
3. Materials used in making piles—	
133,333 pounds cement, at 46 cents per hundredweight (less $120 credit on bags returned)	$493.33
286 pile shoes	1,305.50
50,351 pounds steel rods at 0.0195	981.85
144 cubic yards, at $3.50	504.00
70 cubic yards broken brick, at $3.72	260.40
64 cubic yards sand, at $3.00	192.00
Royalty on concrete piling	644.80
Hauling sand, gravel, steel and cement	415.52
	$4,797.40
Cost materials per foot piling	.74
4. Total cost of pile per foot, items 1, 2, 3	1.13

B. Placing, driving and lining up piles, exclusive of hire of pile-driving plant:

Number of piles driven, 284.

Driving and lining up piles	$1,730.08
Storing piles	141.52
Moving to driver	178.58
	$2,050.18
Cost per pile	7.22
Cost per foot piling	.32

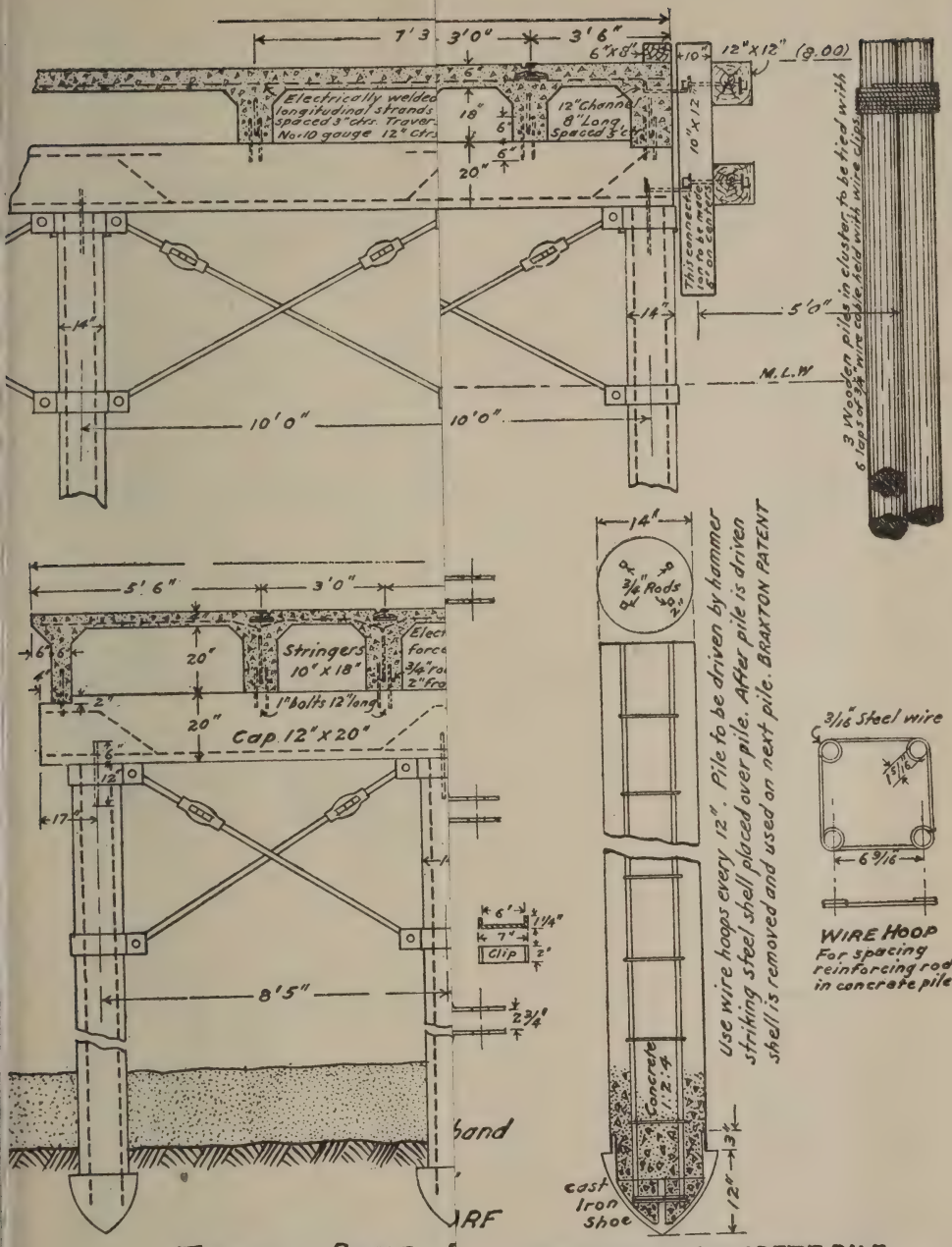
C. Pile-driver plant:

Number of piles driven, 284.

Land driver built by hired labor; one floating driver with lighter and yawl leased at $200 per month; 1 launch leased at $150 per month.

Land driver	$551.74
Floating driver, 2 26-30 months	573.33
Launch	246.78
Coal, oil, waste, etc.	51.95
	$1,423.80
Cost per pile	5.01
Cost per foot piling	.22

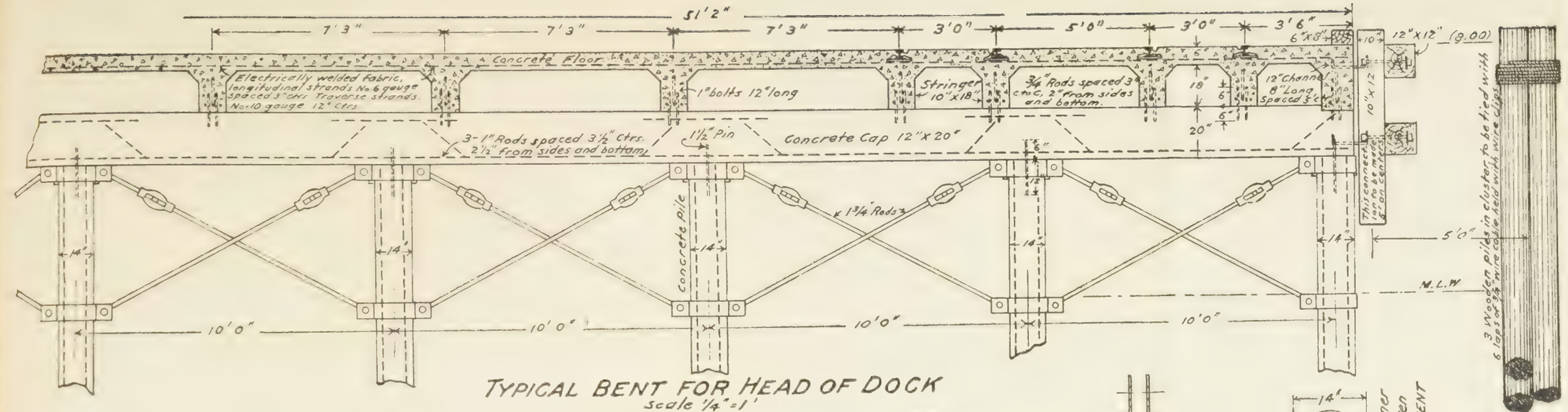
Maximum number of piles driven in one day, 12. Much delay in driving piles caused by rough weather and by obstructing débris from old work.



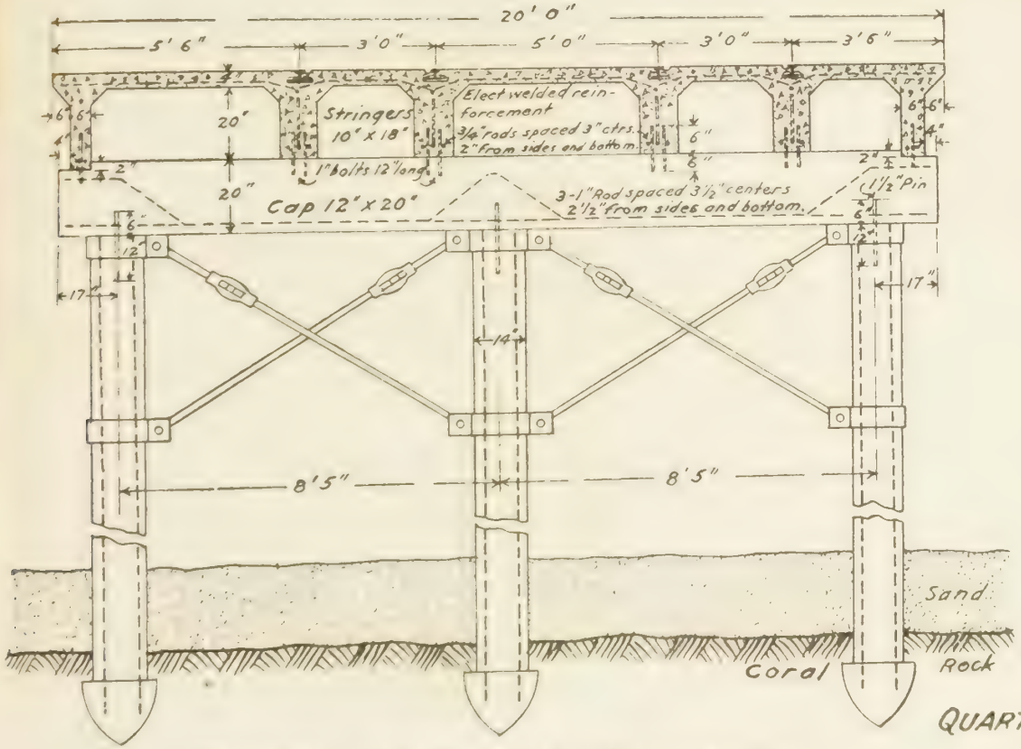
TYPICAL BENT F... SECTION OF CONCRETE PILE

Scale 1/4 of ...909. Scale 1/2"=1'

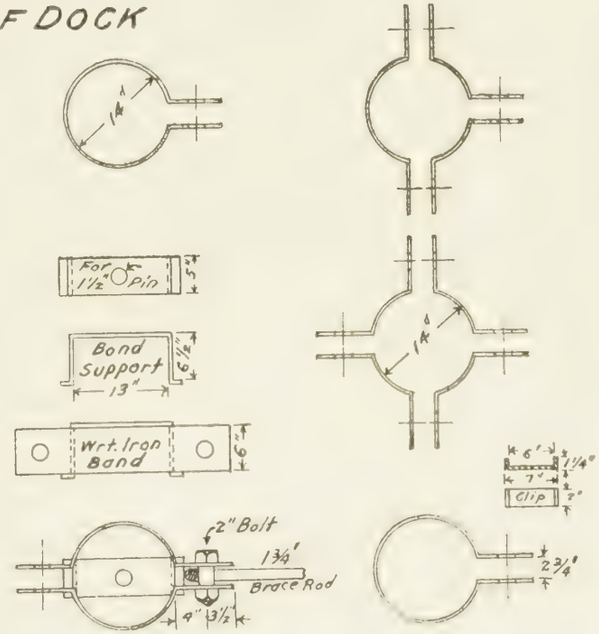
PLATE V



TYPICAL BENT FOR HEAD OF DOCK
Scale 1/4"=1'

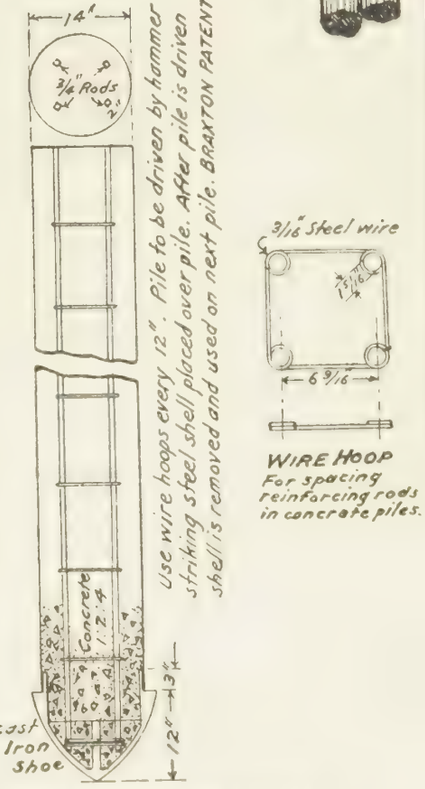


TYPICAL BENT FOR APPROACH
Scale 1/4"=1'



PLAN
Pileband and Support
Scale 1/2"=1'

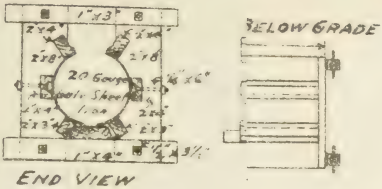
PLAN
Lower Pileband with Clip
scale 1/2"=1'

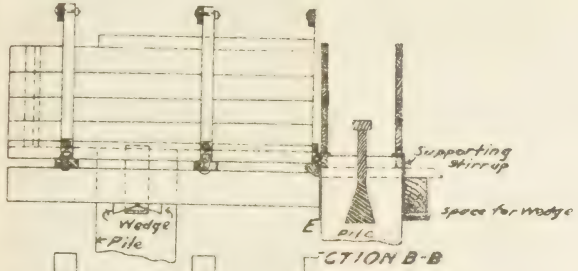

14'
3/4" Rods
Concrete
cast Iron shoe
12"
3"
Use wire hoops every 12". Pile to be driven by hammer striking steel shell placed over pile. After pile is driven shell is removed and used on next pile. BRAXTON PATENT
3/16" Steel wire
6 9/16"
WIRE HOOP
For spacing reinforcing rods in concrete piles.

SECTION OF CONCRETE PILE
Scale 1/2"=1'

QUARTERMASTER AND SUBMARINE MINE WHARF
KEY WEST, FLORIDA
Designed under the direction of GEO. R SPALDING, Capt. Corps of Engineers. By J. M. BRAXTON, Asst. Engineer. February 27, 1909.

PLATE V



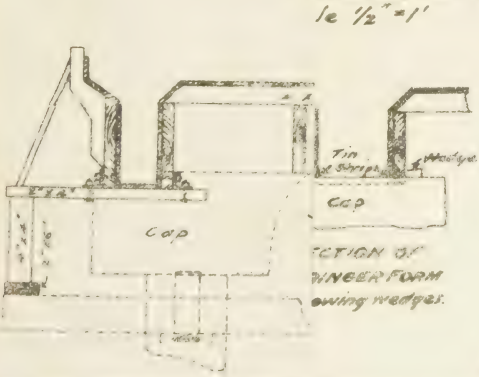


CTION B-B

S OF FORMS
FOR
ETE WORK
& SUBMARINE MINE WHARF
EST FLORIDA

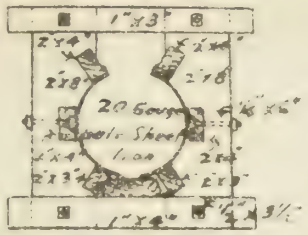
ion of GEO. R. SPALDING, Capt. Corps of
Asst. Engineer. February 27, 1903

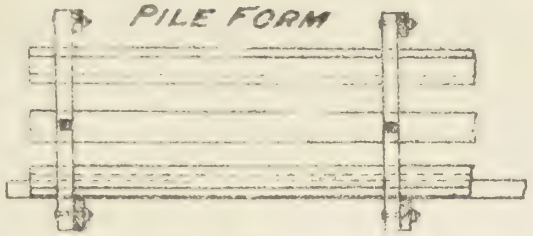
le 1/2" = 1'

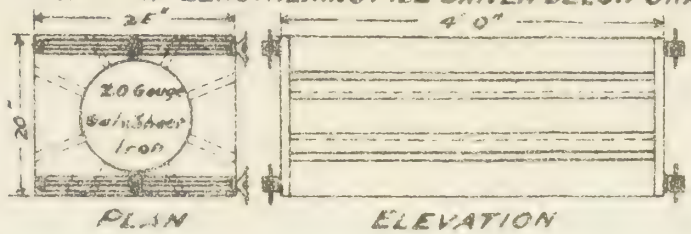


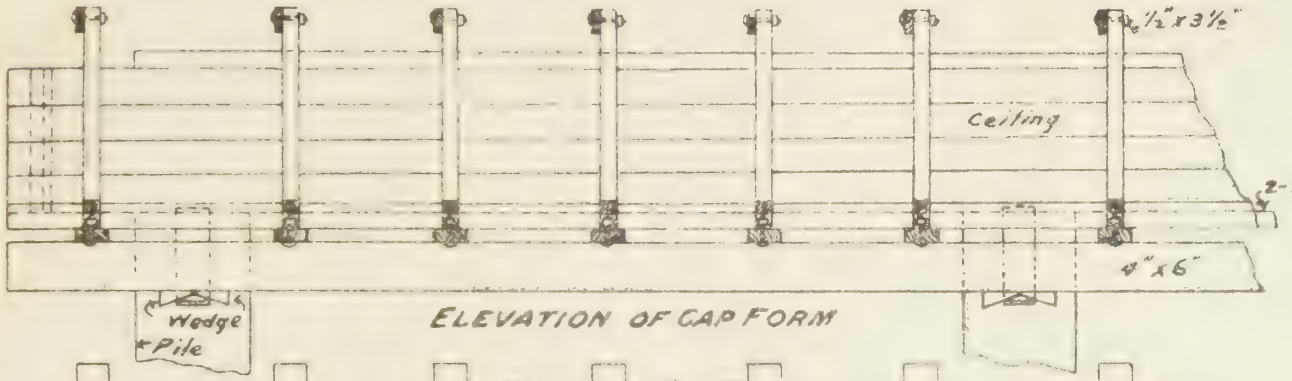
CTION OF
WINGER FORM
owing wedges.

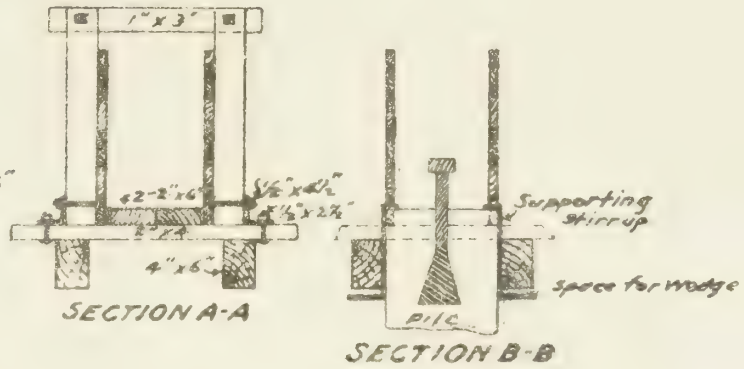
PLATE VI.

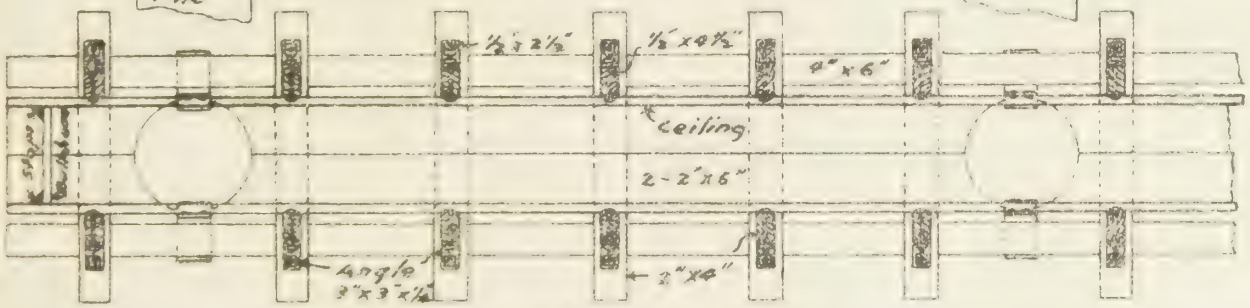








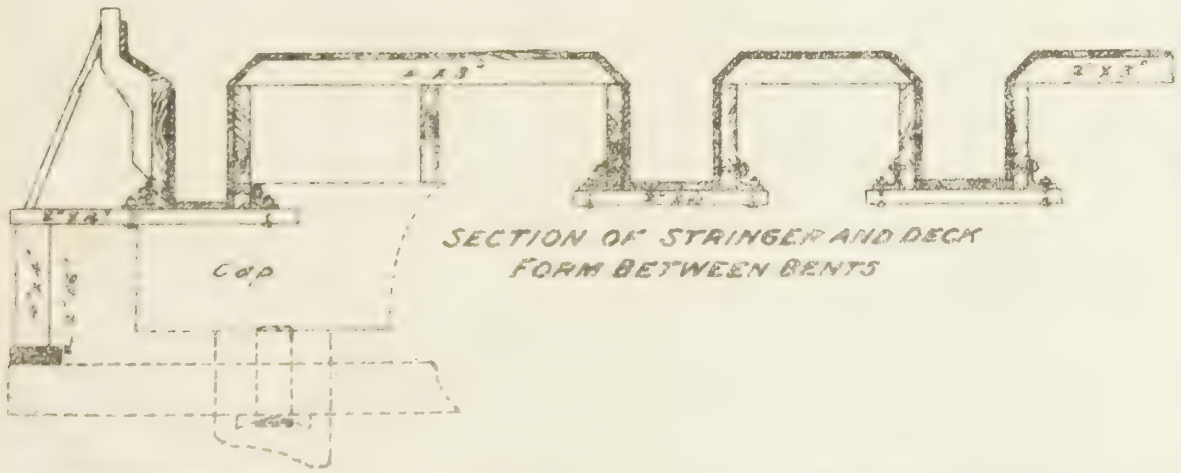


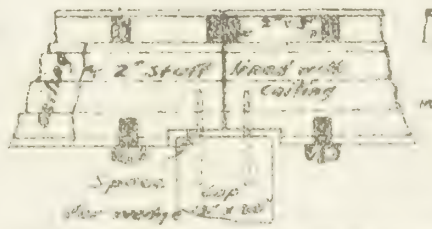


DETAILS OF FORMS
FOR
CONCRETE WORK
QUARTERMASTER & SUBMARINE MINE WHARF
KEY WEST FLORIDA

Designed under the direction of GEO. R. SPALDING, Capt. Corps of Engineers. By J. M. BRAXTON, Asst. Engineer. February 27, 1909.

Scale ½" = 1'





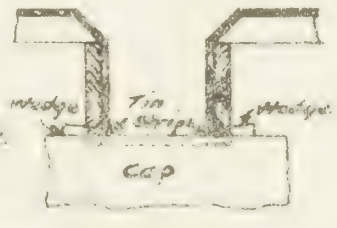


PLATE VI.

D. Pile bracing:

Material, rods, pile bands, bolts, etc., 190,842 pounds, at $0.055		$10,496.31
Hauling materials		136.24
Placing braces		1,521.66
		$12,154.21
Cost per pound in place		.0637

E. Reinforced concrete in caps and stringers:
Total number cubic yards placed, 411.

1. Forms—

Material, lumber, bolts, nails, etc.	$521.32
Making, placing and removing	3,231.27
	$3,752.59
Cost forms per yard, concrete	9.13

2. Materials for concrete—

Cement, 256,667 pounds, at 46 cents per hundredweight (less $194 credit on bags returned)	$986.67
Steel bars, 109,079 pounds, at $0.0195	2,127.03
Gravel, 287 cubic yards, at $3.50	1,004.50
Sand, 128 cubic yards, at $3.00	384.00
Hauling materials	511.78
	$5,013.98
Cost materials per cubic yard concrete	12.20

3. Preparing concrete—

Mixing, wheeling and placing	$1,511.30
Placing reinforcement	245.16
	$1,756.46
Cost preparing concrete per cubic yard	4.27

4. Total cost per cubic yard, items 1, 2, 3 — 25.60

The wharf was designed by Mr. J. M. Braxton under the direction of Capt. (now Major) G. R. Spalding. It was redesigned by Mr. Braxton under the direction of the writer. Mr. George M. Brown was in immediate charge of both the original construction and the reconstruction.

Two Useful Shop Boats.

BY

Maj. H. BURGESS,
Corps of Engineers, U. S. A.

INTRODUCTORY.

The project for improving the lower Tennessee River provides for open channels of 150-foot width and of 6-foot "ordinary low water" depth, and the obtaining of these channels involves extensive excavation of gravel and rock. The removal of gravel is a comparatively easy matter, with the powerful dredges operated by this engineer district, but the extensive rock excavations are slow in progress and expensive in execution. The time for completing the entire improvement of this section of the river is measured chiefly by the rate of rapidity of the rock removal, and the total cost of the improvement will be due about four-fifths to rock and one-fifth to gravel, notwithstanding the fact that there is more than twice as much gravel as rock excavation in the entire project.

It should be mentioned that two attempts were made to let contracts for some of the rock excavation. The estimated cost for the improvement, and upon this the appropriations are made, is based on unit costs for excavation of $2 per cubic yard for rock and 20 cents per cubic yard for gravel, place measurement in each case. For the work at Tuscumbia Bar only one bid was received in response to the advertisement, and this was $2.95 for rock and 45 cents for gravel; and for the Big Bend Shoal work two were received, the low bid being $2.90 for rock and 50 cents for gravel, and the high $11.50 for rock and 60 cents for gravel. At the prices named by the low bidder, in each case, it was not considered advisable to enter into contract, and the method of operating by force-account was adopted. Since it was not economical to construct cofferdams for the purpose of making the rock excavation in the dry, it became necessary to purchase or

construct a large equipment of floating plant, and, for more rapid prosecution of this naturally slow work, two complete outfits were decided on.

There are only six shoals where rock excavation is required, all in the 54 miles between Florence, Ala., and Hamburg, Tenn., and of these six, but four are of great extent. These four are:

Tuscumbia Bar, Buck Island Shoal, Kogers Island Shoal, and Big Bend Shoal; and some of the details of the improvement at each are as follows: Tuscumbia Bar, 95 per cent completed; total length of excavated channel, 1.9 miles; rock cut throughout; 230,000 cubic yards of flint and flint-limestone, and 55,000 cubic yards of gravel. Buck Island Shoal, 97 per cent completed; total length of excavated channel, 2.3 miles, of which about 1 mile is rock cut; 92,000 cubic yards of limestone and 200,000 cubic yards of gravel. Kogers Island Shoal, 25 per cent completed; 2.9 miles of excavated channel, of which 2.7 miles is rock cut; 100,000 cubic yards of flint and flint-limestone, and 45,000 cubic yards of gravel (estimated; surveys not in sufficient detail for exact determination). Big Bend Shoal, 72 per cent completed; length of excavated channel, 3.1 miles, of which 2.9 miles is rock cut; 255,000 cubic yards of limestone and 86,000 cubic yards of gravel.

The equipment of party No. 1, working now at Kogers Island, includes two dipper dredges with 1½ cubic-yard buckets, one towboat and one small steam tender, three derrick-boats equipped with orange-peel buckets of 1 cubic yard capacity, two full-sized drill-rafts and tenders, one half-sized drill-raft and small tender, sufficient scows, barges, quarter-boats, launches, and skiffs, and one carpenter-shop boat and one machine-shop boat.* The last two boats are the subject of the present article. The two full-sized drill-rafts operate twelve drills each, and the two rafts work in echelon along the channel line, each covering one-half the width. The drills are supplied with steam from a 90-horsepower boiler mounted on the drill tender. One of the tenders is equipped with a No. 2 Leyner drill sharpener and a compressor for operating it, this sharpener being able to keep both rafts supplied with sharp bits. The half-sized raft mounts four or five drills, which are supplied with steam from a 40-horsepower

*The method of conducting the rock drilling was described by Mr. J. E. Hall, Junior Engineer, in PROFESSIONAL MEMOIRS for September-October, 1913 (Volume V, No. 23, pages 545-558).

boiler, this raft being used for drilling high points not blasted to grade by the first passing over.

Party No. 2, now working at Big Bend Shoal, has one 2-cubic-yard dipper dredge and one 1½-cubic-yard dredge, one 2-cubic-yard orange-peel dredge, one derrick-boat with a 1 cubic-yard orange-peel, two towboats, two full-sized drill-rafts and tenders, a suitable complement of scows, barges, skiffs, and launches, one or two quarter-boats, and some temporary quarters on the river bank. On the bank, also, are a blacksmith shop and a machine shop, of temporary character and with limited equipment, and for minor repair work only. The drill-rafts mount twelve drills each, and one of the tenders is equipped with the sharpening outfit described above. For important repairs this party send to the machine shop of party No. 1, and the mechanics employed by the latter party are sometimes sent to party No. 2 to help install parts which have been repaired by them at the floating shop. When work was first begun the two parties were sufficiently close to each other to permit even minor repairs to be attended to by the mechanics attached to party No. 1, but the two parties are now separated by about 36 miles distance, and hence the necessity for the small repair force now installed on the river bank at Big Bend.

With the equipment above described, it is hardly necessary to say that frequent repairs of all kinds are needed to keep the various machines at work; that the repairs must necessarily include many kinds of carpenter and blacksmith work, and quite a variety of machine work; and that skilled mechanics are needed for these three classes of repairs. Even if the more important repair work be sent to the nearest commercial repair shop, the mere installation of the new or repaired parts demands the services of blacksmiths, mechanics, and carpenters. However, in view of the distance to the nearest commercial repair shops, it is impracticable to send to them parts needing repair without great resultant delay in the getting of the machine to work again. Of course, when repairs involve work of complicated character, requiring special machines, it is sometimes necessary to make use of the machine shops at Florence or Sheffield, and to send the parts to be repaired by special boat to one of those points; but for ordinary repair work, and in fact for most of it, the equipment and personnel of the carpenter and machine shop boats suffice. It being necessary, for the purpose of making repairs, to install and

operate the shops mentioned, it has been found advantageous and economical also to manufacture small articles needed for repair of machines, etc.

While one of the boats is, for convenience, designated as a "machine shop," it in reality includes a machine shop, a small ice plant, and a blacksmith shop. The reason for installing the ice plant was the high cost of ice at the nearest factory, and the large wastage during transportation to the working parties at considerable distances away. It was found economical to purchase and operate a small plant for furnishing ice to parties Nos. 1 and 2, and to such other Government forces as happened to be within easy reach of delivery.

CARPENTER SHOP BOAT.

The hull of this boat is a decked barge of the gunwale type, 120 feet long, 26 feet wide, and 5 feet deep at the gunwale, with an over-hang deck 2 feet wide. On the barge is constructed the shop building, 106 feet long, 24 feet wide, and 11 feet high at the eaves. The draft under ordinary working conditions is 26 inches. The barge was constructed in New Orleans, at the boat-yard of the Fourth District, Mississippi River Commission, and cost, including towing to Sheffield, Ala., $4,375. It contains 49,000 feet, board measure, of grade "B" yellow pine, with 14 pounds of creosote per cubic foot. The cost of material for the trussing and the cabin was $520 and the labor connected with their construction cost $420, making the cost for the shop boat without equipment $5,315. The equipment was taken in part from an older carpenter shop located at Sheffield, and in part was procured from the shop belonging to the Government boat-yard at Muscle Shoals Canal; and the small tools are mostly the property of the carpenters. The entire cost of the shop-boat and its equipment was approximately $6,420, including installation.

The power is furnished by a 10-horsepower gasoline engine, which drives a woodworker, a lathe, a bandsaw with adjustable table, and a grindstone. There is one electric borer equipped with ship-auger bits (cost, including bits, $225), but the power for it is supplied by the dynamo on the machine shop boat. The carpenters' benches, vises, timber rollers, etc., were taken from the old shop. At one end there are two derricks for handling heavy timbers, small launches, and skiffs, one of which is rigged

with a double and triple block, manila line tackle, and the other with a differential chain block. The derricks place heavy pieces on the timber roller cars, by which they can readily be moved about in the shop. The floor space in the shop is sufficient for simultaneously working on two or three of the longest and largest spuds used in the fleet; for building at one time several skiffs; or for repairing a launch at the same time that skiffs are being built. The uses of the smaller rooms require no explanation. It may be mentioned that for drilling long pieces of steel as, for example, the reinforcing angles on the spuds, the power drill is brought from the machine shop into the carpenter shop, where

Outside View of Carpenter Shop, Showing Derrick.

there is sufficient room for long pieces, whereas these pieces could not conveniently be worked on in the more restricted space of the machine shop. When the demand for carpenter work is light, or when it is mostly done out of doors, the floor space of this shop is largely used for storage purposes.

Four carpenters are employed continuously, and others are sometimes taken on when the amount of repair work is temporarily large, or when a barge or scow is being constructed by the carpenters attached to party No. 1, as is sometimes done when they can not conveniently be built at one of the regular boatyards. The character of work required of the carpenters includes the preparation of extra spuds of various sizes and the

attachment to these of the necessary steel fittings; the preparation or repair of dipper arms for the two dredges; the repair of the small decked boats, thirteen of which compose the large drill-rafts, and which are frequently injured by falling rock after a blast; the repair of the cabins of the several steamboats, dredges, quarter-boats, etc., which also are often injured by far-thrown pieces of rock; the building of skiffs and of the small unit boats of the drill-rafts; the redecking of barges, and especially of scows, or their more extensive repair; the construction of

View of Interior of Carpenter Shop, from Derrick End.

tables, cabinets, etc., for the several boats, and the repair of their furniture; the preparation of patterns for castings needed by the machinists for their work; and, generally, it may be said that nearly all kinds of carpenter work are occasionally required. Much of this work is, of course, done in the open, but the cutting of the timbers and lumber for the out-of-door work is mostly done in the shop, while nearly all work on spuds, dipper arms, skiffs, launches, patterns, etc., is performed within the shop.

Usually there are kept on hand for each dipper dredge an extra dipper arm, one extra forward spud and one extra

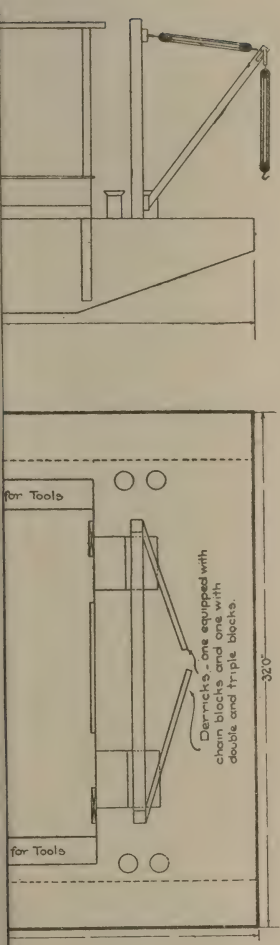
stern spud; for each derrick-boat, at least one extra spud; and there are also kept on hand a supply of the small spuds used for the drill-tenders and for the drill-rafts. So soon as a spud or arm is broken, one of the "extras" is installed and the broken one is turned over to the carpenters to be dismantled, repaired, or utilized for some other purpose; and they and the smiths immediately prepare a new piece to replace the "extra" which was substituted for the broken one. If it were attempted to prepare the arm or the spud after a break had occurred, the delay or the idleness of the piece of plant would extend from one or two days to nearly a week, depending on the character of the broken piece; whereas the keeping on hand of the extra arms and spuds permits the time of idleness to be reduced to a few hours for the spuds and to less than a day for a new arm.

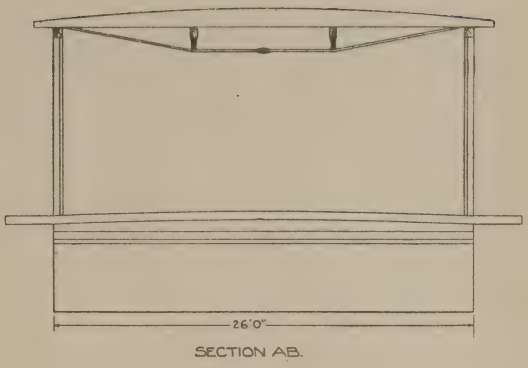
There are attached drawings for this boat—one a deck plan, showing the lay-out of equipment, and the other showing a side elevation and a transverse section. The photographs are external and internal views of the boat, the titles of which are sufficiently clear to make further explanation unnecessary.

THE MACHINE SHOP BOAT.

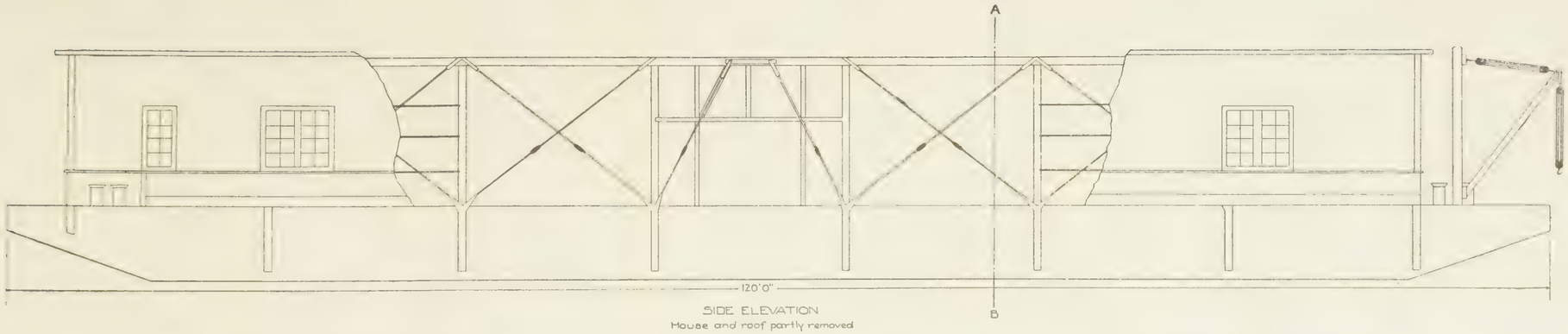
The hull of this boat is exactly like that for the carpenter shop, and was constructed at the same time and place, at a cost (delivered at Sheffield) of $4,375. On this hull is constructed a house 115 feet long, 24 feet wide, and 13½ feet high at the eaves. The over-hang deck is 3 feet wide. The draft of boat under ordinary working conditions is 39 inches. The cost for material in trusses and cabin was $650, and for labor on same was about $500. At one end is the blacksmith shop, and at the other the boiler and two small rooms for workmen. Adjoining the blacksmith shop is the ice plant, and the remaining space is occupied by the machine shop. There are shown in the drawings a deck plan, including the lay-out of the shafting and machinery, and side elevation and a transverse section. The photographs give exterior and interior views of the boat. The total cost for the boat and its equipment is about $11,520.

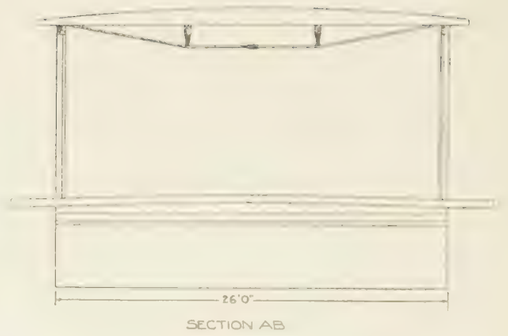
The steam for operating engines, pumps, and generator set is supplied by a 60-horsepower fire-box boiler costing, delivered at Sheffield, $600. It is set up in a room, with good partitions between it and the machine shop and workmen's rooms, in order

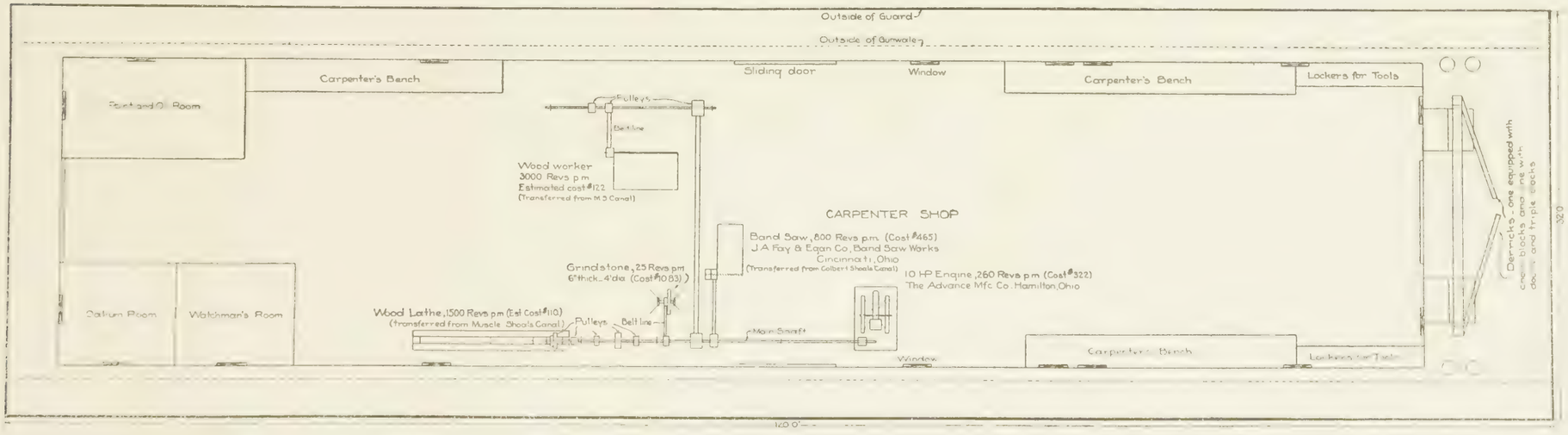




Plan, Elevation and Section of Carpenter Shop.







Plan, Elevation and Section of Carpenter Shop.

to exclude from the latter as much of the heat as possible. The steam supply pipes for the engines and pumps are not shown, but it may be mentioned that these pipes are attached to the lower chords of the roof trusses, and that there is nothing different from usual practice in their installation.

The Blacksmith Shop.

The blacksmith shop occupies the full width of the cabin for 37 feet of length; contains two forges, two anvils, one power blower for the forges, and the usual equipment of tools, quench-

Machine Shop Boat.

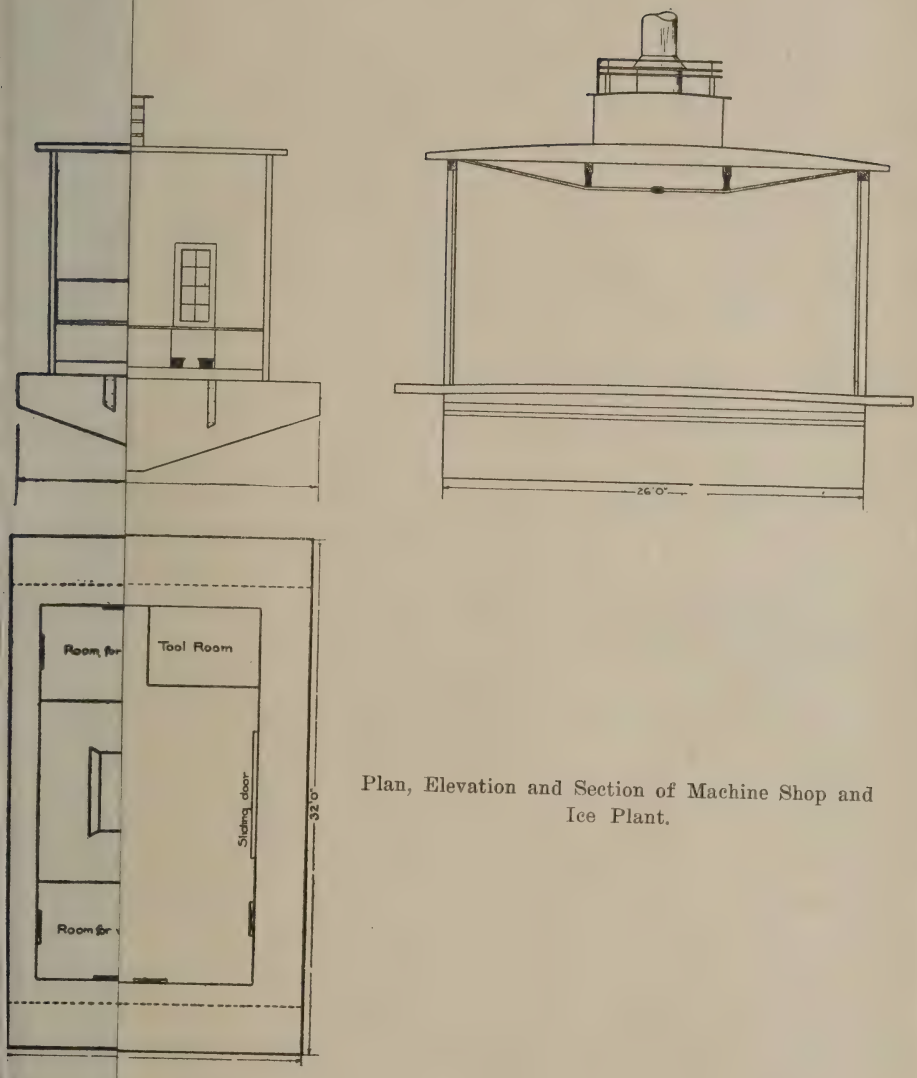
ing tanks, etc. The floor is covered with about a 2-inch thickness of mixed coal dust and cinders, which is kept damp to prevent its catching fire. Three smiths are constantly employed, the chief smith working from 7 a. m. to 4 p. m., one assistant from 3.40 a. m. to 12 m., and the other assistant from 12 m. to 8.20 p. m.; the actual hours of service being eight for each man. A blacksmith's helper is employed for each smith; and the two forges are operated eight and sixteen hours, respectively, for almost the whole of the year.

The competency of the force is shown by the fact that the

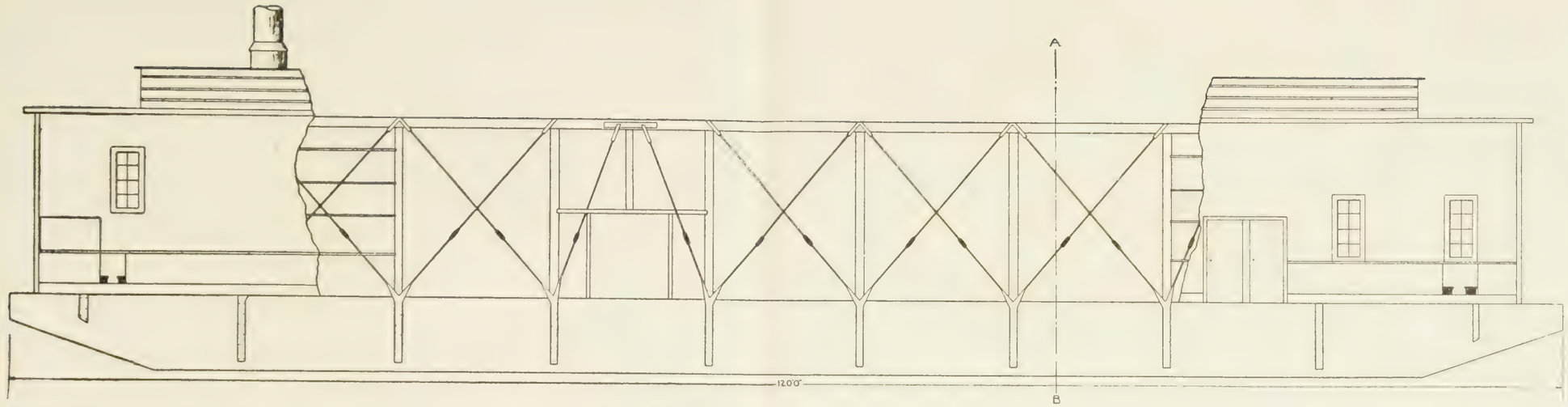
heavy orange-peels and the buckets of the dipper dredges are repaired as well as could be done in the factory, and that the heaviest weldings are successfully undertaken whenever necessary. Recently the wheel-shaft of one of the towboats was broken, and the blacksmiths successfully welded the break in this 5-inch steel, and had the wheel again in service in about forty-eight hours. One of the most important duties of the smiths is to keep ready for use an extra bucket, of each size and shape, for installation on any dredge or derrick-boat. The buckets of the dipper dredges wear very rapidly, especially as to the lip, the teeth, and the front. These parts are made of manganese steel, but, notwithstanding this, the lips and teeth will last hardly six weeks in heavy rock work, and the fronts not more than three months. Extra pieces for the buckets are kept in stock, and there is also one extra bucket of each size, and as soon as the lips and teeth of the bucket in service are too much worn, the bucket is unshipped and the "extra" is installed. The worn bucket is turned over to the smiths, who set to work immediately to cut away the worn lips and teeth, or the front if required, to fit and rivet up the new parts, and to put the bucket in condition for service. A new bucket is purchased occasionally to take the place of one which becomes too much worn to be economically repaired. This method of keeping an extra bucket always ready for installation avoids any further delay than that required for making the change, and having the worn buckets repaired at the shop saves a great deal in cost as well as in time over what would be necessary to have the repair work done at a commercial repair plant. When not engaged in repair work, the smiths manufacture the "blanks" for bolts, nuts, washers, etc., and other small articles.

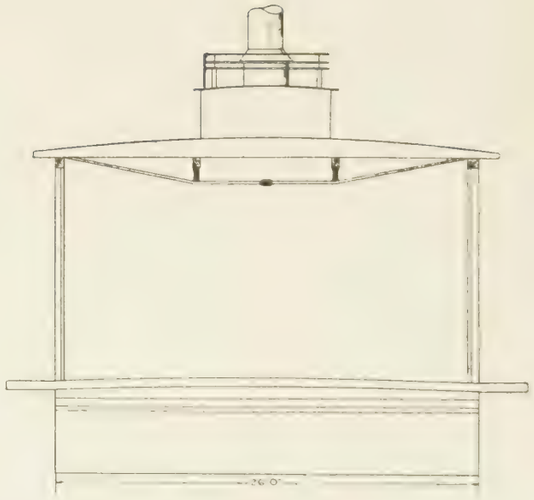
The Ice Plant.

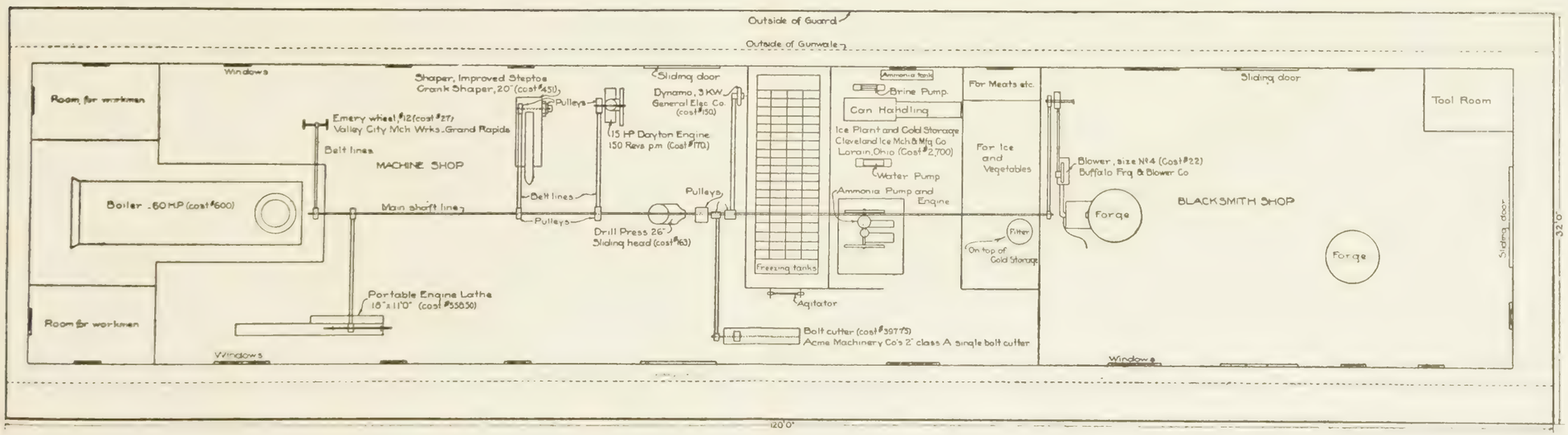
The ice plant occupies 24 feet of the length of the cabin and 18 feet of width; and the entire equipment is installed in this space, except the cooling pipes, which are mounted on the roof of the boat, and the brine agitator, which is placed just outside the partition. The location of the ammonia tank, the ammonia pump, the brine pump, the brine agitator, the cooling-water pump, the filter, the freezing tank, the cold storage room, and the can-handling platform, are all shown on the plan. The cold storage space is 400 cubic feet, and this is used for preservation



Plan, Elevation and Section of Machine Shop and Ice Plant.







Plan, Elevation and Section of Machine Shop and Ice Plant.

of meats and fresh vegetables for party No. 1, as well as a temporary storage for a few ice blocks. For a larger accumulation of ice for shipment to other working parties, there are ice boxes installed on other boats. The rated capacity of the plant is 2 tons per day, including the equivalent of the cold storage room; but the actual capacity permits the manufacture of 2 tons per day, in addition to keeping the room at proper refrigerating temperature.

Since the boiler must be kept fired for operating the machine shop, the cost for steam is divided between the ice plant and

Machine Shop. Showing 15-Horsepower Dayton Engine on Left; Shaper in Foreground; Drill Press in Center and Bolt Cutter in Right Background.

the machine shop, which makes the ice production somewhat cheaper than if a separate boiler were required. The cost of labor for the ice plant amounts to about $160 per month, and the cost of steam, ammonia, oil, and other supplies is about $110, making the cost per month for operation about $270. Although the daily amount of ice, plus cold storage, is more than 2 tons per day, that figure is taken because that is about the amount actually needed during the hot months; and, thus, the cost per ton is $4.50, not including interest on cost of plant nor deterioration.

The ice purchased in the season preceding the installation of this plant cost $8 per ton delivered at the Sheffield landing, and amounted to about 2 tons per day for the hottest months and about 1 ton per day for late spring and early fall months. Allowing for wastage in shipment and for transportation to the working parties, the cost per ton would now be not less than $10. The saving in cost of ice for each season is probably not far from one-fourth to one-third of the cost of the plant.

The Machine Shop.

This shop occupies the space on both sides of the boiler room and on one side of the ice plant, and the full cabin width between the ice plant and the boiler room. The power for operating the machines, the dynamo, and the blower of the blacksmith shop is furnished by a 15-horsepower "Dayton" center-crank, vertical engine, costing $170, which drives the main shaft shown on the plan. This shaft is attached by suitable brackets to the lower chords of the roof trusses, and the general layout of the shafting is about what it would be in a small machine shop on land. The machines are: A General Electric direct-current dynamo, 3 kilowatt, 110 volt; an engine lathe, 18 inches by 11 feet; a 26-inch drill press, with sliding head; an improved Steptoe crank shaper; a bolt cutter for threading nuts and bolts; and an emery wheel. Three mechanics are constantly employed, the arrangement of hours of duty for the chief and the assistants being the same as for the smiths. The repair work demands that this number of mechanics be employed; but at times, they would be idle, except for the manufacturing of drill parts, bushings, etc., as mentioned hereinafter.

The principal work required of the mechanics is the keeping ready for service at all times of the full number of drills needed for four full-sized and one half-sized rafts; that is, about fifty-two to fifty-four drills. The service of the drills, especially in the flint, is quite severe, and breaks are numerous, and the result is that four or five drills per day come into the shop for repair. The extent of drilling necessary for the four shoals mentioned above is shown by the fact that there are required to be drilled for the four over 210,000 holes, aggregating something over 1,250,000 linear feet of drill hole. In order to keep the required number in service there have been purchased seventy-three drills, which permits about twenty-two or twenty-three to be in the shop

at one time without interfering with the work. Experience has indicated the parts which most frequently break or are worn out, and a considerable stock of these parts is kept on hand, so that overhauling of a drill may be quickly effected. These parts are ordinarily purchased from the manufacturer of the drill, but occasionally minor parts are fabricated in the shop.

The machinists, when not engaged in repair work, make up bolts, nuts, fusible plugs for boilers, extra shipper-shafts for the dipper dredges, extra bushings for all bearings on engines and dredges, keys for shafting, pulleys, shafts for the dredge engines, and many other steel or cast iron parts and fittings. Rough castings are obtained at the canal foundry, blanks for bolts and nuts from the smiths, shaft-steel and other unfinished parts by purchase from dealers. Many of the small articles are fabricated at costs as high or higher than the price at which they could be purchased; but when it is remembered that the machinists must be employed anyway, it is apparent that there is a saving in having such articles made at the shop. However, ordinary articles are made only when there is a slackening in the repair work.

The following list shows the actual cost of making certain dredge parts at the shop, and the price at which similar articles were purchased from dealers within the past two years. To the shop costs there should be added about 10 per cent to cover depreciation, overhead charges, etc.

Article.	Shop cost.	Cost from Dealer.
Dipper dredge parts:	*Dollars.*	*Dollars.*
Dipper shaft, weight 50 pounds	2.25	5.00
U-bolt, weight 125 pounds	6.50	13.50
Latch keeper, weight 60 pounds	4.75	9.80
Latch bar, weight 36 pounds	2.80	5.94
Bottom band, weight 200 pounds	8.50	13.35
Back brace, weight 108 pounds	4.60	8.69
Door plate, weight 345 pounds	14.60	18.98

DISPOSITION OF THE BOATS.

So soon as the work of improvement on the lower Tennessee River has reached that degree of completion which makes unnecessary the use of an extensive equipment of floating plant and of drill outfits, it is expected to dismantle these shop boats and

to use their equipment at the two regular boat-yards pertaining to the Tennessee River improvement. The lumber of the cabins can be used for repair of the houses on derrick-boats or dredges, and the barges themselves will be useful for all purposes for which that class of craft is needed. Were it not for the necessity for moving the two working parties from one shoal to another, it is probable that the installation of the shops on the two boats would not have been advisable, as there could have been constructed at less expense temporary shops on the river bank in the vicinity of the shoal under improvement. Under the necessity for changes of locations for the two working parties, however, it is believed that the two shop-boats will prove to have been more useful and less expensive in the long run than temporary shops on land, which must be torn down and re-erected from time to time as the parties are moved to new work.

Discussion: Vertically Framed Mitering Lock Gates

BY

Brig. Gen. H. F. HODGES,
United States Army.

The writer has read with interest the valuable paper by Mr. Malcolm Elliott, printed in the PROFESSIONAL MEMOIRS for July-August, 1915, on "Vertically Framed Mitering Lock Gates," and hopes that it will have the effect of directing attention to this type of structure.

Regarding the discussion as restricted to metal gates, the writer has no doubt that the vertically framed type possesses great merits, the chief being the certainty with which the stresses in the different members can be computed, and the simplicity of the framing, due to the fact that there are few intersections of main members. The latter advantage is very apparent in single-sheathed gates. In large double-sheathed gates, with air chambers, the framing becomes usually more complex; but even in these, the cells into which the gate is divided, by the main vertical frames and the few horizontal diaphragms, may be made of shape permitting convenient access for examination and repainting; while in cellular gates, with the main frames horizontal, the compartments are usually so low that men find it difficult to work in them effectively. The simplicity of construction tends to make the unit cost of the vertically framed leaf usually less than that of the horizontally framed leaf of any but the simplest and smallest description; and, for this reason, will ordinarily justify the adoption of the former type, even when the dimensions are such that horizontal framing would give a somewhat lighter structure.

The contention of the author that, for a girder gate, the advantage will remain with the vertically framed leaf when the height is less than the length, is probably justified as a general guide. The exact point of equality in weight of the two types will change somewhat with the loading considered, and with the amount of vertical and diagonal bracing necessary to give rigidity to the leaf, as well as with the design of the girders.

The point at which theoretical equality will take place may be determined without difficulty for given conditions. Thus, assuming that the leaf retains the full height of water on one face and that the lower pool is empty to sill level, the load per unit of

length of the horizontals will be $\frac{\delta h^2}{2}$ and the total load on the leaf $\frac{\delta h^2 l}{2}$, h being the height above the sill, l the length and δ the weight of a unit of volume of water. In case of a horizontally framed leaf, this load will produce on the combined horizontals, supposed to carry it to the end posts, a total bending moment of $\frac{\delta h^2}{4}(lx-x^2)$, x being the distance of the section from either end taken as the origin. Assuming the unit resistance to tension or compression to be c and the depth of the girders between flange centers to be D, then the volume of the compression flanges, or of the tension flanges, proportioned at all points exactly to the endured stress, will be found by multiplying the expression for the bending moment by $\frac{1}{c\mathrm{D}}\,dx$ and integrating between $x=0$ and $x=l$.* The result is $\frac{\delta}{c\mathrm{D}}\frac{1}{24}h^2l^3$ for either the tension or the compression flange. The volume of both flange systems is double this expression. The maximum shear on the entire horizontal system is $\frac{\delta h^2 l}{4}$. Assuming the unit resistance to shear to be s, the aggregate web area is $\frac{\delta}{s}\frac{h^2 l}{4}$. If the girders be built with flange area varying with the endured stress and webs of uniform thickness, proportioned to resist the greatest shear, the weight of the horizontals will be measured by the expression

$$\frac{\delta}{c\mathrm{D}} \times \frac{1}{12} h^2 l^3 + \frac{\delta}{s} \times \frac{1}{4} h^2 l^2 \qquad (1)$$

the first term measuring the volume of all the flanges combined, and the second term that of the webs. In the vertically framed gate the weight of the framework is made up of the main upper girder, carrying one-third of the load over the span l between the posts; and the vertical girders, loaded triangularly and spanning the height h. The volume of the main girder, supposed with uniform web and varying flanges, is one-third that of the horizontal system already found. The vertical system, supposed of the same depth, D, as the horizontal system, may be assumed to consist of many girders of uniform cross-section, proportioned with aggregate flange-area sufficient to resist the maximum bending moment, viz, $\frac{5}{81}\,\delta h^3 l$,

*Neglecting the longitudinal compression, which increases one flange and decreases the other by about the same amount.

and with aggregate web-area sufficient to resist the greatest shear, viz, $\frac{2}{3}\frac{\delta h^2 l}{2}$. The aggregate volume of the vertical girders will therefore be measured by the expression

$$\frac{\delta}{\mathrm{CD}} \times \frac{10}{81} h^4 l + \frac{\delta}{s} \times \frac{1}{3} h^3 l$$

and the total volume of the vertically framed gate by the expression

$$\frac{\delta}{\mathrm{CD}} \left(\frac{1}{36} h^2 l^3 + \frac{10}{81} h^4 l \right) + \frac{\delta}{s} \left(\frac{1}{12} h^2 l^2 + \frac{1}{3} h^3 l \right) \quad (2)$$

the first term measuring the volume of all the flanges, and the second that of all the webs. The expressions for the volumes of the webs in (1) and (2) will be equal when $l=2h$; while the expressions for the flange volumes become equal when $l=1.49h$. The skeletons of the leaves will therefore be equal in weight, theoretically, for some value of h lying between 1/2 and 2/3 of l if the girders be of the assumed construction.

If the horizontal girders be assumed as having unvarying flanges proportioned to resist the greatest bending moment, viz, $\frac{1}{16} \delta h^2 l^2$, then the expression for the volumes of the flanges of the horizontally and vertically framed system will be found to be equal when

$$\frac{\delta}{\mathrm{CD}} \left(\frac{1}{8} h^2 l^3 \right) = \frac{\delta}{\mathrm{CD}} \left(\frac{1}{24} h^2 l^3 + \frac{10}{81} h^4 l \right)$$

or, when $l=1.22h$. In this case, therefore, the skeletons will be equal in weight, theoretically, for some value of h lying between 1/2 and 5/6 of l.

The foregoing brief analysis is the foundation for the statement made by the writer, and cited by the author, that leaves of the horizontally and vertically framed systems will have the same weight when the height is about 2/3 of the length.

The conclusion is evidently approximate only, on account of the assumption stated and of others not specifically mentioned. Thus, the flange area is regarded as dependent upon the transverse stress alone, neglecting the area required to resist compression in the lower flange near the ends. Further, all horizontally framed leaves have some vertical rigidity, and generally transmit a considerable part of the load to the sill directly, with corresponding relief to the horizontal girders. If advantage be taken of this relief in the construction, and the horizontals be correspondingly lightened, a skeleton will result which will be lighter than that given by the horizontal girders proportioned to carry the entire load. Furthermore, a vertically framed leaf should have some horizontal rigidity, in order to make tight contact at the miter and quoin posts.

Neither of these considerations has been taken into account in the above analysis, but both should work to make the ratio of height to length, at which the weights balance, slightly less favorable to the vertically framed type. Also, the assumptions take no account of the sheathing; nor of the vertical and horizontal members which are almost always introduced, respectively, in horizontally and vertically framed gates, as stiffeners and bulkheads; nor of the diagonal braces. All these have been assumed to equilibrate each other, and may or may not do so. As a net result, the writer believes, according to the theory, that the height must be somewhat less than the length to make the two systems equal in weight. Nevertheless, considering the other advantages which the vertically framed structure has to offer, he believes that, in the general case, it will be found economical in first cost, for gate leaves in which the length is not less than the height. Conditions other than immediate economy, however, may dictate the use of another type.

Thus, when the load is very great, the question of the local stresses involved, in delivering the pressure at the points of support, must be considered before arriving at a determination. Also, the probability of structural damage to the leaf, from obstructions caught between the miter posts, or in the hollow quoin, must not be left out of sight. The horizontally framed leaf promises the better resistance to such accidents of service.

With reference to the author's remarks, page 411, on the depth of the gate recess for horizontally framed leaves, this need not be excessive, even if the upper girder be very deep. The member may be dropped 2 or 3 feet below the top of the gate, and the part of the gate above it may be built, as a relatively narrow foot-walk, on supports resting on the downstream part of the web, and sheathed to hold back water above the level of the girder. The masonry may be corbelled out in such manner as to extend over the upstream part of the girder, when the gate is in its recess. The coping will then be narrowed only by the width of the foot-walk, instead of the full depth of the girder. Unless the overhang has to sustain heavy weights, the construction will not add greatly to the cost of the masonry.

The author makes a comparison between the calculated weight of the vertically framed leaf, designed for the lower gate of the new Louisville lock, and the actual weights of leaves of about the same size, built for the Mississippi River lock at Keokuk, and for the Panama Canal. In order to clear up any misunderstanding, it should be observed that the horizontally framed type was selected for the Panama Canal because the majority of the leaves are so high with respect to their length that the vertically framed type was out of the question. Of the ninety-two leaves in the canal, seventy-two are from 82 to 66 feet high, inclusive, eight are 57 feet 8 inches high, and 12 are 47 feet 4 inches high, measured from the level of the top of the miter sill. All are 64 feet 10 inches long between surfaces of contact at the end posts. Only twenty,

therefore, are decidedly well adapted, by shape, to vertical framing.

It would have been possible to build the low gates with vertical frames, and the remainder with horizontal frames; but such a course would have entailed additional and special drawings, templates and details for the gates, anchorages, masonry, and sills, and would have neutralized, in part, the economy which it was hoped to effect through uniformity of construction of the main members of all the gates of the canal. With so large a number, standardization promised to result in saving. On the whole, it was thought better to make these smaller leaves like the others, rather than to save part of their weight by a radical change in the design. The low price at which the gate contract was let, viz, 3.875 cents per pound for the structural parts, erected, is an indication that the course taken was right.

It should be remembered also that, as provision against corrosion, the plates used were, in places, somewhat heavier than the stresses required. No sheathing plates less than 7/16 inch, and no web plates less than ½ inch in thickness, were permitted, thus adding weight not needed to resist the static stresses. The 47-foot gates are undoubtedly considerably heavier than they would be, if vertically framed; but the gates of the canal, as a whole, probably cost less than if a special design had been adopted for the low ones.

It may be stated that these gates are built to carry the full depth of water against the face, when the pool behind them is empty. The author does not state the conditions of loading for which the Louisville lower gates were designed. The load naturally affects the weight of the leaf.

With reference to the author's remarks on the use of buoyancy chambers (page 413), it may be stated that there are in existence examples of large metal leaves with curved backs, built without air chambers, which function well in service. The curved shape does not necessarily preclude bracing, nor does it require an air chamber to prevent distortion of the leaf. In the writer's opinion, the buoyancy has its greatest value in lessening the weight on the fastenings during operation, rather than in preventing distortion, against which other provision can usually be made.

There are certain examples of large vertically framed gates, which the author might have cited to support his opinion that such can be built. The locks of the Trans-Atlantic Dock and the Bellot Basin at Havre are closed by vertically framed gates. Each leaf of the former is 56 feet 7 inches by 34 feet 9 inches in elevation and weighs 175 tons; of the latter 53 feet 4 inches by 35 feet 9 inches in elevation, weighing 171 tons. The structures are described at length in the *Annales des Ponts et Chaussées* for 1887 and 1889, respectively; and, briefly, in *Professional Papers, No. 26, Corps of Engineers, U. S. A.*

The framework of a gate, consisting wholly of vertical girders

between the top girder and the bottom diaphragm, can not be expected to possess the rigidity of the framework of the better-known type, in which the main members are horizontal, and are bound together by a number of verticals. Nevertheless, the former type possesses rigidity enough for its work, ordinarily. In two cases, of which the writer has knowledge, vertically framed metal gates have shown a surprising resistance to shock. The accidents occurred at the Trans-Atlantic Dock at Havre, and at Lock No. 1, Big Sandy River. In the first case, the gates were opened and shut several times by waves when the pools were nearly equal. Ultimately, an anchorage broke and let one leaf fall over into the lock. In the second case the leaves, which were fastened back in their recesses in flood time, were wrenched loose and slammed violently together by the current. Neither case resulted in serious damage, the leaves being readily repaired and put back to work.

The writer has long favored the use of vertical framing for gate leaves where the dimensions are favorable for economy, and other conditions are favorable for endurance. He proposed gates of this type for the lock at Herr Island on the Alleghany River in 1892, and a few years later introduced such gates on the Big Sandy and Kentucky rivers. A drawing of the Big Sandy River gates will be found in the Annual Report of the Chief of Engineers for 1902, facing page 1948. Nevertheless, he believes that instances will occur where the horizontally framed type is to be preferred, even though the dimensions are such as to make a vertically framed leaf the lighter. The extra weight of the horizontally framed leaf, if disposed in such manner as to add to its rigidity and resistance against shock may, in some cases, be an advantage, rather than a detriment.

As for the Ohio River gates, the dimensions point clearly to vertical framing, if the miter gate be built. It will be of great interest to note, in practice, how the new miter gates compare in efficiency with the rolling gates formerly adopted for the locks. Some of the earlier "freight-car gates" were in themselves not so satisfactory as they might have been, apart from the objectionable gate recess, which the author mentions.

Duty, Organization, Training, and Equipment of Engineer Troops for Field Service.

BY

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The duties of engineer troops in the United States Army in time of war may, with respect to location, be classified as follows:

1. At bases, mobilization camps and advance supply depots.
2. On the line of communications.
3. In the attack and defense of fortified places (siege operations).
4. With the mobile army in the field.

Each of these is further capable of sub-division, and may include a great variety of works. Duty with the mobile army is the prime function of engineer troops, and is the one on which their organization, training, and equipment should primarily be based. All other engineering duties (except possibly siege operations) must be regarded as special. They will, when necessary, be performed by engineer troops, either regular or volunteer, with such changes or additions in personnel or equipment, civilian assistance, etc., as the special situation or exigency may demand.

The engineer troops for field service should be with the combatant forces at the front, at all times subject to the immediate orders of the commanders of the divisions, corps, or armies to which they may be attached. Their duties will be intimately connected with the movements and tactical operations of the fighting forces, and should be characterized by extreme rapidity, full use of local resources, and a thoroughness all sufficient unto the immediate needs and *no more* than sufficient. Subject to these conditions, the operations of the field engineers may be roughly classified as follows:

1. Operations to facilitate the rapid movement of the combatant forces.

2. Operations to increase the offensive or defensive powers of the combatant forces and to limit or decrease those of the enemy.
3. Operations to maintain the health and promote the comfort of the troops.

The works required under these heads will be numerous and varied according to the course of the campaign, the nature of the theater of operations, the season, the weather, and other conditions. The most extended catalogue of such works would still be incomplete, but a statement somewhat in detail of the more important duties ordinarily to be performed, is necessary as a basis for our discussion.

It is of the utmost importance in every case to draw the line of demarcation between the duties of the field engineers at the front and those which properly pertain to the engineer or other personnel on the lines of communication. This is necessary in order to insure smooth cooperation, and the presence in every tactical emergency of the engineer troops with the combatant forces. The supreme commander in the field will in each case prescribe and limit the functions of the two organizations. Those of the field engineers should include no work which might have been performed by others. There will be enough which they alone can execute, and their energies should not be frittered away on work for which other special troops or civilians are or might be made available.

The more important special duties of engineer troops in the mobile army may include:

1. Reconnaissance of the natural and cultural features of the terrain, preliminary to tactical operations, or for other purposes. Reconnaissance of hostile works and dispositions.
2. Collection of maps and other data from local sources.
3. Correction and amplification of existing charts.
4. Mapping of limited portions of the terrain within the sphere of tactical operations, and other minor survey duties.
5. Map reproduction, field methods.
6. Collection and utilization of local engineering resources in personnel and matériel.
7. Laying out of defensive positions and points of support.
8. Planning and superintendence of offensive or defensive field fortifications, including obstacles, sapping and mining, etc., and the execution of the more difficult tasks in connection therewith.
9. Laying out and improving camps.
10. Construction and repair of roads, railroads and bridges.

11. Construction of temporary buildings, and repair of permanent buildings and other structures.
12. Military demolitions, etc., etc.

The ideal engineer troops are those prepared to execute all tasks which may be assigned them, and the engineers of the United States Army are trained with this end in view. In the field they must not only meet, but anticipate the needs of the Army. To this end, the chief engineer should be at all times in close touch with the commander-in-chief and with the engineer troops, and each battalion and company commander of the engineers must be at all times in close touch and hearty cooperation with the troops of the line with whom he is serving.

On account of the variety of duties exacted of an army in war, we find a strong tendency in many countries to specialize the work of the technical troops. By this I mean that certain organizations are trained to perform a certain kind of work, and are not supposed to be employed in anything outside of their specialty. Thus we find in the European armies sapper companies, railroad companies, aero companies, telegraph companies, etc. One foreign engineer has even recommended the organization of water supply companies.

In our own service the duties of the foreign aero and telegraph companies fall to the Signal Corps. All the remaining technical operations demanded by the modern army are to be performed by the engineers, an entirely feasible arrangement. If we were to have special troops for each technical duty, there would be no limit to their numbers. Moreover, many of these duties are sporadic in their nature and the troops trained to perform them would necessarily be idle a large part of their time, or else, which is more probable, they would be assigned to other duty, for which they should have been trained. So far as our own service is concerned, a multitude of special troops would be a useless expense, and our engineer companies can and should be trained to perform satisfactorily all the engineering operations required by the mobile army, including the mapping of limited portions of the terrain, the construction of temporary buildings, water supply, the hasty repair of railroads, etc. A single organization, large enough to have representatives with every command from a brigade up, is thus always available and qualified to perform any task that may be assigned. With special organizations for each class of work, the number of each would necessarily be relatively small. It would

be impossible to assign them to any except large commands. They would frequently, as a consequence, be wanting when most needed, would be idle part of the time and swamped with work during the remainder. It would be better to have the necessary special workmen and plenty of "handy men" in each company, with officers qualified to superintend and direct all classes of technical work. Engineering operations with the field army are nearly all in the nature of makeshifts. They should be sufficient for the immediate purpose, and no more. Extensive work of a special nature (excepting siege operations) will generally be executed in rear of the army, and special provisions therefor can be made as the occasions arise.

The officers of the Corps of Engineers of the United States Army are, as a result of their combined civil and military training, qualified to plan and direct all of the engineering operations which would be required of them in time of war, and it is on them that our chief reliance must be placed.

Mapping work over large areas can be carried out by engineer officers and troops, assisted by details of qualified officers from the line. It is desirable, though not essential, that officers so detailed should have had some previous experience in survey work. Extensive railroad and building construction can be performed by engineer officers and troops, preferably, of course, by those who have had some previous experience on such work, either in the army or in civil life, of whom a number will always be available. These troops may be assisted by a few expert civilians, who would usually be drawn from the regular personnel of the railroad, or the operations may be performed entirely by civilians, preferably under contract. The conduct of such works will present no insuperable difficulties.

The engineering of siege operations demands no special training other than that which all engineer officers and troops normally receive. The addition to the engineer personnel so engaged of a few individuals skilled in mining operations, preferably enlisted, would, of course, be desirable.

The organization of special companies for ponton work is, in the writer's opinion, not only unnecessary but unwise. All engineer companies are now trained to perform this work, and a company of trained infantry, under the supervision of engineer officers, can be made proficient pontoniers in a month. If there is to be any specialization it should be in survey or railroad work.

A precise military survey of a large area in the Philippine

Islands was successfully conducted by a company of engineers which had received no special training for such work prior to its assignment to the task. This company also constructed its own barrack, a large frame building. Another engineer company located, built, and operated a first-class railroad, without any previous training in such work for either officers or men, constructed many buildings, some of them of concrete, permanent wagon roads, works of land fortification (concrete) and many other structures. A battalion of engineers constructed a highway bridge of ten 90-foot spans with 18-foot roadway on Cushing piers, across a large unfordable river, subject to flood. There were no civilian assistants and the plant was very inadequate, consisting of a single-drum hoisting engine and an orange-peel bucket (which was operated with one drum). Neither officers nor men had any previous experience in the construction of large bridges. The examples mentioned are some that have come under the writer's personal observation quite recently. Many others might be cited. In the British service we find the Royal Engineers successfully erecting a great variety of engineering structures. With such examples before us, it is believed we need entertain no fear that our regular engineers will not be found equal to any of the exigencies of a campaign.

On the basis that each engineer unit shall be trained to perform all of the duties which may by regulation or necessity be demanded of engineer troops in campaign, the question of organization is greatly simplified.

The company is essentially the administrative and technical unit of engineer organization. In the United States Army companies are further organized into battalions of three companies, one battalion being assigned to each division of the mobile army. There is no higher organization at present, although regimental organization is advocated and has advantages. The present organization permits the temporary assignment of one company to each of the three brigades in a division, if desired, and for service with mounted troops mounted engineers are provided by regulation, although companies with full mounted personnel have never been organized.

Division of the companies, except by their own officers, is not advisable. Splitting the engineer companies should generally be avoided by the higher commanders, and in all cases should be for tactical rather than technical purposes. If an engineer officer is to be held responsible for the prompt and thorough performance of his technical duties, he should be allowed to distribute his own

command in the manner best calculated, in his opinion, to carry out the tasks assigned him. The mounted section of a company may, however, properly be assigned, temporarily, to duty with the cavalry, advance, rear, or flank guards, when a need for its technical assistance can be foreseen. It should never be assigned for mere escort duty when other mounted troops, cavalry or scouts, are available.

The number of officers with a company is not prescribed. Due to the need for expert superintendence in its various technical duties, and the fact that the company will frequently be divided when engaged therein, it will be apparent that the number of officers should be greater than in an infantry organization of the same strength, which would seldom be divided, and whose duties are less varied in their nature. Judging by experience, it is believed that the number of officers with an engineer company in our service should never be less than five, and need seldom be more than seven.

We come now to a consideration of the enlisted personnel:

The present authorized war strength of an engineer company (pioneers) is 164 enlisted. This accords fairly well with the practice of foreign armies, and fulfills the following conditions:

1. It enables the officers of the company to be intimately acquainted with the qualifications and character of each individual soldier. Were the size of the company much greater this would hardly be practicable, in view of the frequent changes of personnel, both officers and men.
2. It forms a strong working force, capable also of subdivision into a number of parties, each strong enough to do effective work.
3. It permits the tactical employment of the company as infantry, which is one of the duties contemplated in our regulations.

It is believed that there should be in each company at least three or four men corresponding to the grade of petty officer in the Navy. These men should be paid from $75 to $100 per month, in addition to their allowances, should be designated "pioneer sergeants," or better, "master pioneers," and should have the qualifications of first-class foremen of construction in addition to their military training. These men would fulfill the duties of overseers or foremen in immediate charge of gangs engaged in construction work. They should be able both to lay out and execute all ordinary tasks. In addition to supervising the work of the engineer soldiers they could be usefully employed in charge of working details from the other branches of the service (or civilians) and on reconnaissance. They would thus fulfill many duties which in their absence would neces-

sarily be assigned to engineer officers. While being general utility men, they should also specialize along certain lines, such as topographers, experts in road and bridge work, field fortifications, railroad track work, etc. They should be familiar with simple building construction and concrete work. In the operations of field engineering, the company would often be divided into squads or gangs of twenty-five men or less. If working independently, each of these gangs would require an overseer. It thus appears that the number of master pioneers with a company should be at least four, and in view of their wide range of usefulness and probable employment away from the engineer troops, six per company would not be too many. With the present pay and allowances there is no assurance that men with such qualifications will be present in a company.

No successful contractor would think of undertaking an important job without a sufficient number of skilled foremen, and the Government should make provision for men of this class with the engineer companies. One good foreman, capable of independent action, and resourceful enough to meet emergencies, is worth to the captain more than twenty unskilled men. In fact, in a pinch he may even be worth a whole company of such, just as Stonewall Jackson's presence on the field was worth a reinforcement of 50,000 men to the Confederacy. Labor is usually cheap and plentiful. In an emergency the commanding general can supply additional labor by the detail of troops from the combatant forces or the hire of civilians, which latter should generally be avoided except in extreme cases. But the engineers will be expected to furnish skilled superintendents, of which they should have an adequate supply.

The personnel of the company should consist of foremen, artisans of various classes, skilled and unskilled labor. The artisans need have only the regular trades of civil life, very little additional technical training being required to adapt them to military uses. The following should be included in the personnel:

General foremen,	Cooks,	Photographers,
Track foremen,	Blacksmiths,	Lithographers,
Carpenters,	Machinists,	Clerks,
Riggers,	Horseshoers,	Engine runners,
Teamsters,	Plumbers,	Concrete workers,
Packers,	Transitmen,	Miners,
Farriers,	Levelmen,	Quarrymen,
Saddlers,	Sketchers,	Electricians,
Shoemakers,	Draftsmen,	Woodsmen, etc.

With plenty of helpers and laborers.

The number of men of each trade will depend on its importance from the point of view of military exigency. Thus, for instance, there can hardly be too many carpenters. At least fifteen or twenty journeymen are desirable, and every man in the company should be a "jack-leg"—able to handle a saw and hammer. On the other hand, one shoemaker with an apprentice in training will probably be sufficient. Furthermore, it is neither necessary nor desirable that all these trades be represented by distinct individuals. The trades have a tendency to overlap more or less, and the all-round or handy man is exceptionally valuable. Thus a good machinist is frequently also a blacksmith and a plumber, and may in addition to this be a carpenter and know how to run an engine. A good bridge carpenter (the cabinet type of woodworker is not desired) must of necessity be a foreman and a rigger. It is to be remembered that the fine points of these trades find no application to the rough-and-ready work of field engineering. A plumber who can cut, thread, and joint pipe will meet all requirements. He will not be called upon to make a "wiped joint." Hence it is not too much to expect that many of the men will possess the necessary proficiency in more than one trade. It is not always essential that the men shall have previous practice in the special tasks that may later be assigned to them. A handy man will always acquit himself creditably even on a task unlike anything he has ever before encountered. For example, a machinist with a good general experience can be trusted to do simple plumbing work in an entirely satisfactory manner, even if he has never before threaded a pipe. He has certainly threaded a bolt.

The bulk of the company will, in general, consist of men who would rank as "helpers" and laborers, although such titles are not applied to them. These men should be given opportunity to learn trades for which they show an aptitude, and from their ranks many clever artisans will be developed.

Special mention should be made of the cooks. The cook is an important artisan and the one most to be praised or, according to the nature of his seasoning, damned. He is the only one for whom a title and special salary is now provided by law. The importance of the cook can not be exaggerated. Good food, well prepared, promotes health, cheerfulness and discipline. A poor mess creates universal dissatisfaction, and may more than offset all the tact and disciplinary skill of the officers. At present, two cooks only per company are provided for by law. Considering the size

of an engineer company, and the frequent necessity for splitting it into widely scattered detachments, this number is manifestly insufficient. Four regular cooks should be provided, and in addition a considerable number of men should be instructed in cookery.

The entire personnel of an engineer company should be enlisted men, subject to the control of the captain.

The pay of the enlisted personnel of the engineer troops should be greater, grade for grade, than that of the other troops of the line. When a larger organization offers equal pay and demands less arduous work, it is difficult to secure satisfactory men for a smaller organization. With a higher rate of pay the nature of the service would attract to the engineers the class of men specially qualified for service in their ranks.

The mobility of an engineer company should be somewhat greater than that of the division to which it is assigned, for in addition to keeping up with the march of the column it has also to perform its special work, and may be called upon to fight. Accordingly, provision should be made for relieving the men of their packs whenever practicable. It has often been proposed also to relieve the engineers of their rifles, too, or at least to reduce the number of rifles to the company, replacing them by revolvers. They would thus become strictly noncombatant troops. I believe this would be a mistake. So long as the engineers retain their rifles they have all the fighting power of the best infantry, and, even if they be not actually employed on the firing line, form at all times a not inconsiderable reserve to the combatant troops, enabling the commander to commit more troops to action than would otherwise be practicable. We have the word of a great soldier that a single battalion may turn the tide of battle. To deprive the engineer soldiers of their fighting power would to a great extent deprive them of their self-respect and spirit, thereby reducing their efficiency. The engineer soldier now feels that he is equal in all respects to any man on the field. It does not seem wise to deprive him of this sentiment, nor to deprive the Army of his fighting strength by making him a mere laborer instead of a skilled artisan and a good fighter. However, the engineers should normally carry on their persons less ammunition than the infantry, and the bayonet might well be dispensed with.

The movements of large bodies of troops, except during actual combat, will be confined to the roads and railroads, and consequently most of the engineering operations will be performed

near to these highways. However, there will not infrequently be occasions when a detachment of engineers must be sent ahead or to the flank, possibly across country, for a considerable distance, for the rapid execution of some relatively small special task, such as the destruction of a bridge or the interruption of a railway. To meet such demands mounted men are required, and each company in the United States service is, accordingly, provided with a permanent mounted section of twenty-four men, which greatly increases the territorial range of their operations. Each man can carry his rifle and one or two tools, as well as some material (such as wire or explosives) in his saddle bags.

For service with a cavalry command it is manifest that an engineer company must be mounted throughout. For service with foot troops it does not seem to be advisable to mount the entire company, as the necessity for taking care of the horses would greatly decrease the working capacity of the organization. The mounted section now provided appears to be sufficient to meet all the ordinary demands of service with foot troops.

For the rapid transportation of tools and materials in section work, particularly the work of the mounted section, some means of transportation of relatively small capacity and of greater mobility than the standard army wagon (in which the equipment is regularly carried) is desirable. In the German service this need is met by small one or two-horse carts, capable of moving across country. In the United States service the same end is attained by the use of a small pack train. As to which of these methods is the better, the nature of the terrain, number and condition of the roads, etc., will decide. In the United States the pack train is, I believe, ordinarily to be preferred. The normal assignment is six pack mules per company. In case a greater number is required they can be transferred temporarily from another company, or from the division pack train. For service with a cavalry command, or in very mountainous country with poor roads, the entire equipment of the company might be carried on pack mules.

No standard pack equipment for the engineers has ever been provided, and the need for suitable frames, capable of carrying the standard tools, and readily loaded and unloaded, is very great. Experiments should be undertaken for the purpose of deciding upon a suitable equipment. The pack animals regularly assigned to the company should then be furnished with the standard engineer equipment. Animals temporarily assigned from the division

train will generally need nothing in addition to their regular rigging.

Some officers favor a tool wagon (on standard running gear) provided with compartments, to facilitate issue of the equipment—sometimes called a "pie wagon"—which, they claim, has the added advantage of keeping the tools in better order. Others prefer to carry all tools in the regular army wagon and make issues from piles on the ground. Which of these methods will be the more satisfactory depends on the nature of the equipment, the terrain, and the individual preference of the officer concerned. Delicate instruments should certainly not be thrown in at random with picks and shovels, but the number of delicate instruments should be a minimum. Saws, chisels, adzes, bits, etc., should be in light boxes of a size and shape suitable for packing on a mule. Transits, compasses and the like should, of course, be in cases. Such equipment may be placed in the front of the wagon under the seat separated, if need be, by a transom from the rougher tools in rear. Each method has its advantages and its advocates. Were the equipment absolutely unvarying, it is probable that the "pie wagon" would be the more satisfactory means of conveyance. As the equipment is not standard, there is something to be said for the other method. In any case, the running gear of the wagon should be that standard throughout the Army.

The end and aim of all organization, training and equipment is to produce the maximum of efficiency in the field. In an article by an Austrian engineer officer, the writer encountered this statement: "The efficiency of engineer troops resides primarily in their tools and equipment." This is like the statement that the fighting power of the infantry soldier resides chiefly in his rifle and ammunition, and seems a narrow argument as the basis of a discussion on the training of engineer troops. There are moral and intellectual forces which go to make up the necessary equipment of the field engineer. They are of more interest and more importance than inert tools and materials.

An officer to command engineer troops should have the following qualifications:

It is essential that he have a good constitution, able to withstand the hardships of a campaign. He should be a man of high professional qualifications, both as a soldier and as an engineer. Without such qualifications an engineer officer can not properly plan nor execute the varied works that will be assigned to him, nor properly

subordinate his work to the needs of the combatant forces; he can not command the respect and confidence of his own men and can not, consequently, maintain discipline. He should be zealous in the performance of his duty—which ordinarily implies an interest in his work, resourceful, and last but not least, a student of human nature, knowing how to handle men so as to produce the best results under trying circumstances. The non-commissioned officers should possess qualifications similar to those of the officers, although not necessarily so great in degree or scope. Each, within his own sphere, should be capable of intelligent, independent action and command.

The enlisted men should be both workers and soldiers. Their training along both lines and their discipline should be of the best.

Discipline is the first requisite of a soldier. It is that quality which chiefly distinguishes the army from the armed mob. It is of an importance in engineer organizations at least equal to that in other arms. It will be of interest, therefore, to inquire into that of which discipline consists and the methods by which it may be promoted.

Discipline has been variously defined as "a spirit of cheerful obedience," "a virtue which impels us to carry into effect the intentions of our chief through a sense of duty, and for the general welfare, by devoting to this end the whole physical, moral and intellectual energy of which we are capable," etc. The best indication of the state of discipline in a command; in fact, the actual test thereof, is the manner in which the men perform the tasks assigned them. There are also some external signs of discipline—chief amongst which are cheerfulness and alacrity. Sullenness amongst the men is ordinarily indicative of a bad state of discipline. However, seriousness, even grimness, should not be mistaken for sullenness. In the midst of privation, suffering and death, a diminution in the outward signs of cheerfulness is to be expected, but true discipline will yet prevail.

The discipline of a command—a company—is to some extent dependent upon its circumstances and environment. Chiefly, however, it depends upon the personal influence of the captain. I can think of no better illustration of the attitude which a captain should assume toward his men than that of a careful carpenter toward his tools. The carpenter, finding that his saw does not work properly, stops working and calmly examines the tool to see what is wrong. Having set the teeth, he makes another attempt to

saw. The tool still works badly. So the carpenter lays it aside, and selects another to finish the task. Later, he takes out the refractory saw and patiently works on it until a test shows it to be in good order once more. Or, failing this, the carpenter, after working long over the saw, reflects as follows: "I can not afford to spend so much time on this saw. I am neglecting my other tools and wasting my employer's time." And the saw is discarded in the interests of the carpenter's employer and for the benefit of the other tools.

The captain is responsible for the discipline and training of the company. The men are the tools of the Government, placed in his hands. It is his duty to maintain them in good condition and to use them with intelligent care. Human tools respond more readily to good treatment, and are far more easily injured by bad than mechanical tools. The captain should be a first-class civil and military engineer, and a thorough soldier, and should be known as such to his men. They are quick to detect these qualifications or the lack of them, and their knowledge of high professional attainments on the part of the commander begets their respect and a confidence not only in the captain, but in the organization which he has trained, and in its individual members, including themselves. The captain must realize that he and his men are working in a common cause. He should be the friend and guardian of his men, doing all in his power to promote their comfort, well-being and happiness. He should be intimately acquainted with each of them, knowing his powers and limitations, and he should display a friendly and personal interest in the work, recreation, etc., of the soldiers without, of course, making unwarranted intrusions into their private affairs. The simplest attentions, which ordinarily will cost the officer nothing, indicate to the individual soldier that his captain takes an interest in him, and this knowledge helps to command his loyalty and devotion. The American soldier is more readily controlled by love than by fear. And while an officer may, by harsh punishments and brutal treatment, secure the appearance of obedience on the part of his men to the letter of his commands, he does not secure their confidence and respect, their cooperation and loyalty. Without these he can not command their best effort. The American soldier responds promptly to good treatment, and quickly resents harshness and brutality.

The command should at all times, both in garrison and in the field, be spared all unnecessary hardships and annoyances, and

should be made as comfortable as the circumstances will permit. Appreciating such considerate treatment, the men will the more readily respond to the extraordinary demands upon their skill and endurance, which will so frequently be necessary in campaign. The captain should explain to his men, so far as may be advisable, the occasion for such extraordinary demands, to the end that they may be brought into sympathy with his purpose and the means he takes to accomplish it. The intelligent American likes to know that there is a reason for what he is doing. He will undertake almost anything if you can convince him that it is necessary.

Frequent changes of personnel, both officers and men, in a company, are to some extent detrimental. But the spirit of discipline once established is readily maintained, unless the company falls into the hands of incompetent or indifferent officers. New men coming to a well disciplined organization ordinarily soon catch its spirit.

In brief, the secret of discipline is to get the men on your side, to make them loyal to you. They will then follow you through fire and water, and with such an organization at your back you may safely venture any undertaking within the power of men.

The efficiency of an engineer company is directly in proportion to the efficiency of and, within reasonable limits, to the number of its officers. Other things being equal, a company of one hundred and sixty-four men with six officers will be nearly twice as efficient on section work as one with three. It is upon the officers that the chief reliance must and will be placed, and their training, especially their early training, is therefore of the first importance.

To properly fulfill his functions in the field, an engineer officer must, as has been stated, be a thorough soldier and a competent civil engineer. His training in both callings should be along moral and technical lines. Military training with engineer troops, under the conditions which now obtain in the United States service during peace times, is not alone sufficient to develop the resourcefulness and technical knowledge of engineering which are so necessary. These are acquired as a result of the additional training on the important civil works with which the Corps of Engineers is charged. The conduct of these great works also gives to the Corps a certain amount of prestige, standing or reputation which promotes esprit and serves also to stimulate the zeal and awaken the pride of the younger officers. This alternation of civil and military work maintains interest and prevents the ennui which some-

times results from garrison work too greatly protracted. The dual peace time rôle of the Corps of Engineers seems to be in every way advantageous.

The training of officers is of even greater importance than the training of the men. Raw troops under efficient officers will rapidly become efficient, but the best of troops in the hands of incompetent officers may be almost useless. Youthful impressions and habits early acquired are difficult to change or eradicate, be they good or bad. Consequently, the most important part of an officer's training is that which he receives early in his career. And the most important feature of this training is not the acquisition of technical experience so much as the development of high ideals of service and devotion to duty. An officer of average intelligence, who is industrious and loyal to his ideals, is usually more valuable than a brilliant man who endeavors to substitute cleverness for zeal and hard work.

This training devolves upon the older officers, who should appreciate that they have no higher duty. The young officers should not be kept in the background, but should early be put in a position to acquire experience. They should be placed, from the beginning, in positions of trust with as great responsibilities as they are capable of bearing. It is responsibility above all else that develops character, initiative and judgment.

A very important faculty which the military engineer should possess is what may be called a "sense of proportion." By this is meant the power to distinguish and separate the important things from those which are unimportant or relatively so. A man who does not possess this faculty is not qualified to make important decisions. Its full exercise in professional affairs requires a knowledge of details, so that an inexperienced man, however great his latent talents, is not qualified to make important decisions, a fact which our Government has often failed to appreciate in assigning men to high military command. But there are many men who, possessing a thorough knowledge of detail, never learn to weigh all the facts and circumstances in connection with a case and classify them in the order of their relative importance, in making a decision. Such men never rise above mediocrity, and if by chance or favor they are placed in important and responsible positions, they become monumental failures. This important faculty is developed by responsibility, by the necessity for making decisions, and it is for this reason that responsibility is the most important feature of

training. It develops a man's self-reliance and gives him confidence in himself. A young officer who is kept in a subordinate position, never permitted or compelled to exercise independent command or authority, is very apt to have his sense of proportion atrophied from want of use.

An impression prevails in some quarters that works of military engineering are conducted without regard to the principles of economy as practiced by the civil engineering profession, and this is explained on the theory that principles of economy must of necessity be disregarded in military operations. This is not true. No builder who disregards the principles of economy in his work is entitled to be called an engineer, and this is true of the military engineer to an even greater degree than of his civilian confrère. Both are judged by results, and the military engineer may have opportunities to commit blunders or to effect economies not only of money, but of national prestige and prosperity and of blood, by comparison with which the greatest opportunities offered to the average civilian engineer sink into insignificance. To avoid these blunders and to effect these economies will often require the keenest perception and the greatest breadth of judgment, which is to say, in a high degree, that sense of proportion to which reference has been made.

The economics may often be quite different in the two classes of works, military and civil, but the same general principles govern both. The cost of a modern war between two great nations is enormous* As a consequence the march of events, as Von der Goltz says, will be rapid in modern warfare. Everything must move with the utmost celerity. The Government pays enormous liquidated damages for every day that the duration of the war is protracted. While a correct decision will not usually be easy, the element of time will ordinarily be the controlling consideration. The commanding general does not ask his engineer, "How much will it cost?" but "How soon will it be ready?" A consideration of the case from the proper economic standpoint will usually indicate that *any* expense is justified if it results in a saving of time at a critical juncture. This, of course, frequently simplifies matters, yet it takes a strong and determined man to disregard utterly money and life in the accomplishment of his purpose. It is a fatal

*The cost of the present European war has been estimated at $50,000,000 per day to all the belligerents.

blunder to hesitate over details involving a few dollars when millions are at stake.

As a consequence of these conditions, works of military engineering will usually be conducted with feverish rapidity. The best pioneer is the one who produces results just sufficient for the immediate purpose in the minimum of time. Formal and finished work will be left to those in rear. The pioneer with the mobile army eagerly seeking a tactical decision has no time for niceties. Their presence in his work is an almost certain indication of criminal incompetence—lack of a sense of proportion. A trail ferry which gets the troops across in time is better than a memorial bridge—which does not. The rough-and-*ready* makeshifts which serve their purpose are the triumphs of the military engineer's art. They have the beauty of utilitarianism.

In practice work in military engineering for instruction purposes in time of peace there is a strong tendency to lose sight of the prime importance of speed. An officer has not necessarily acquitted himself with credit because he has built an excellent bridge across an otherwise impassable stream. If he spends a day preparing his plans and collecting his materials, another day organizing his forces and initiating the work, and two days to complete, with a fifth admiring and exhibiting his work with a company of engineers standing around in the meantime, when the whole job should have been done in one day by fifty men, he has in such case made a most pitiful failure, for he has neither appreciated nor taught the most important part of the lesson. Such instruction may even do more harm than good, in creating false ideas as to the manner in which military engineering works should be conducted. Details must indeed be taught, but in simulating the construction of complete works in the field the element of speed should be placed in its proper relative position—that is to say, FIRST.

The methods by which speed in construction work may be attained are so numerous and so varied, that a thorough discussion of them would fill many large volumes. They constitute an interesting and useful study which no engineer, military or civilian, can afford to neglect.

In war time a stimulus for rapid work will seldom be lacking. It is desirable, however, as has been pointed out, that training in time of peace should inculcate speed in construction. Whenever practicable the work should be planned in advance. The engineer

should rapidly but carefully scrutinize the details of his plans, eliminating all features which add to the time required for construction without being absolutely necessary to the completed structure. The remaining details should then be reduced to their simplest form. The necessary tools are laid out, and steps taken in advance to have all necessary material on hand as soon as it may be required. The work should then be divided into a number of definite tasks under the officers and senior non-commissioned officers, and these again subdivided into smaller tasks, under the junior non-commissioned officers. The cost of superintendence, which might be prohibitive on civil work, is justified in military construction if it results in a saving of time. If several of the tasks can be made similar in nature and equal in amount, so much the better. This will create competition, always a stimulus to endeavor. Two identical structures with an equal number of men assigned to each will usually call forth the best efforts of both parties. Of course, that task which will require longest to accomplish should be started first and pushed hardest, if the time required for its completion is the measure of the time required for the work as a whole. But all odds and ends which might, at the last moment delay the finish should be cleaned up well in advance. The number of men assigned to any task should, as a rule, be the minimum that can properly execute it. Too many men on a task are always a cause of confusion and delay. If there be a superfluity of men, it is better to put them on some other work or send them back to quarters than have them standing idle. Extra men, if kept on hand, can be best utilized in

a. Collecting and preparing materials, or
b. Relieving the men engaged in the more arduous tasks.

It should never be necessary to have any men idle. There will always be tasks in connection with any piece of work, the performance of which will not tire the men as much as standing around, which is very fatiguing. Extra non-commissioned officers should be put to work, on the more dignified tasks, of course. They should not be allowed to get the idea that their sole function is to superintend. The senior non-commissioned officers should not scruple to lend a hand when necessary, or to show some man who is bungling his work the proper method. But they should not in so doing neglect the more important work of superintendence. The captain or officer in supreme charge should watch the work

as a whole, generally leaving details to his subordinates. The foreman of any particular task should, when practicable, be given a specific time in which to complete it, usually the least time in which the task can possibly be accomplished. While he may fail to complete on time, he will naturally be anxious to avoid the explanations that will then be demanded of him. If no time is set for completion dawdling is very apt to result. Two gangs mutually dependent on each other, as where one furnishes the material which another incorporates in the structure, should be so organized that each will be hard pushed to keep up with the other. This is in the nature of competition. If the number of tools is limited, care should be taken that all tools are doing useful work all the time.

The training of individuals should not be limited to one particular class of work. There should be several men trained to perform each kind of special work, the number depending on the relative frequency and importance of that work. Also, each soldier should be trained to perform *well* a number of tasks, according to his aptitude, and should be given *some* instruction in *every* class of work performed by the company. Every man should be a "jack-of-all-trades" and, in addition, as many as possible should be also "masters of one" or more. The natural tendency of the trades to over-lap has been referred to. This tendency should be cultivated and fostered. To this end, at every post where engineer troops are stationed, there should be a trade school conducting regular courses during the season for indoor work, the men of the company being assigned to the various courses of instruction. As soon as a man acquires proficiency in one department he should be shifted to another, generally one in which he has elected to take the course, or for whose work he has shown an aptitude. The trade schools, in connection with outdoor courses in seasonable weather, should provide for instruction in all the regular duties of engineer troops, including drafting, lithography, etc. The instructors would be officers or enlisted men, or in some cases civilians. During the open season a number of men should be trained in foot and mounted reconnaissance and sketching, use of the plane table, transit, level, etc. Such men should, of course, also participate frequently in the general exercises of the company, to the end that their training be not too highly specialized or one-sided.

The object of such training will be obvious. The engineer

company, even when depleted by wounds, disease and death, should still be able to perform all of its functions.

In addition to its special training every engineer company must also be thoroughly instructed as infantry, including simple close and extended order drill, target practice, marching, camping, etc. Having once been thoroughly trained in this way, an occasional drill or maneuver will be sufficient to maintain efficiency and keep up the men's interest. Even an infantry close order drill is pleasant and interesting, if it is not of too frequent occurrence or too long duration. Such drills, with occasional inspections, parades, field exercises, marches and a little guard duty, serve to maintain the soldier's interest and efficiency, and to stimulate his pride in his personal appearance and soldierly bearing and capacity. The engineer troops should not be allowed to degenerate into mere laborers, nor to become slovenly and unsoldierly in appearance. Neither they nor the infantry troops with whom they may be serving should be allowed to entertain the idea that the engineers are mere "ditch-diggers," and not in all respects thorough soldiers. They must have and must prove that they have all the qualifications of the best infantry, and much in addition.

Continuous application to any kind of instruction usually becomes monotonous in a remarkably short time, and dulls the interest. The variety of the engineer's duties affords to the resourceful officer better opportunities to sustain the interest of his men than are afforded to the officer of infantry, the latter having in this respect a more difficult task. Interest in one's work is the chief reason for efficiency. Recruits coming to the organization should be given a thorough course of infantry squad drill and instruction in the simpler details of their engineering work before being assigned for full duty with the company. This instruction will be given by non-commissioned officers under the supervision of the officers. It is monotonous work for the instructor, and the entire burden should not be laid upon one or two men because they have shown special efficiency in such work, but should be distributed amongst a considerable number of the non-commissioned officers and qualified privates, in the same manner as other duties.

Whenever possible, works executed for training should also serve some useful purpose. Some works for instructional purposes only will certainly be required, but if all be of this nature it will be difficult to sustain the interest of the men. The intelligent American can not be expected to take the same interest in work done

"just for fun" as in that which he knows will serve some useful purpose after its completion. When practicable, routine work for instruction only should be varied by the employment of engineer troops under their own officers on some important Government construction work. This will stimulate interest, and being a larger task, will afford a broader field for instruction. The fact that such work may be somewhat different in its nature from what would be required in campaign is no great detriment. The same lessons can be taught. It will seldom be practicable to transport the troops any great distance from their stations in order to perform such works, though it would be well if this could occasionally be done. But at every large post there is construction work on which engineer troops might be employed to advantage. All construction work on posts, as distinguished from ordinary maintenance, supply and transportation, could properly be placed in the hands of engineer officers. With such a provision engineer troops could often be employed on post construction work with great advantage in the way of training, and with economy to the Government.

It is essential that the engineers should be familiar with the requirements of the other arms, and, on the other hand, it is necessary that the other arms should know the powers and limitations of the engineers, to the end that all may cooperate to the best advantage in campaign.

Lack of mutual understanding and knowledge in these matters may have unfortunate results, to-wit: On the part of the engineers it may lead them to fritter away their time in the construction (in the field) of works of minor tactical importance, while neglecting more important duty; and, on the part of the other arms, they may fail to utilize the services of the engineers to the best advantage or, on the other hand, may expect and count upon more than they can accomplish.

This mutual knowledge should be possessed primarily by the officers of the various arms, and it is therefore essential that officers should observe and study the tactics of other arms, and that all should mingle freely in professional and social intercourse. Engineer troops should be stationed in time of peace with troops of the other arms, and not at separate posts. Each should take part in the exercises of the others and all should attend maneuvers together. At maneuvers, the engineers should, as far as practicable, be employed in connection with the tactical operations of the other

arms. This is a case where familiarity breeds cooperation and unfamiliarity breeds mutual contempt and distrust.

For infantry and artillery, as one can foresee the nature of their operations, it is easy to provide a standard equipment. With the engineers, due to the wide range of their activities, the problem presents special difficulties. A complete standard equipment for engineer organizations in the United States Army has never existed, even on paper. Personally, I believe it never will exist *except* on paper. The duties of the engineers are so numerous that to attempt to provide an equipment to cover all contingencies would destroy their mobility, without which any equipment would, of course, be useless for operations with the mobile army. An equipment provided for service in Virginia would be unsuited for the Philippine Islands, and one provided for duty in the field would not be entirely suitable for siege operations. The work of the engineers in the field, while varied, is all of a rough-and-ready order, and simple in its nature. For the performance of this work an ample number of the simple tools should be provided. For example, an adze is a most useful tool in the hands of a man who understands it. But it can not do much which might not have been accomplished with an axe; whereas the axe is capable of much for which an adze is useless. Hence, for pioneer operations, take the axe and leave the adze. Of course, there will occasionally be a call for an adze, so there should be a few of them in the company equipment.

While the proper outfit for a company would never be the same in any two situations, nevertheless it is possible in a general way to decide upon the types and numbers of the more important tools. Lists of such tools would constitute the "paper equipment." A sufficient number should be issued to the troops for their training in time of peace, and a reserve should be kept on hand in a depot, sufficient at least to equip the first troops to take the field in case of war.

The tools and equipment of the engineers may be divided into two general classes, viz:

a. Commercial types of tools and materials.
b. Special tools, apparatus and equipment used only by the army.

Under (a) would be included the great bulk of engineer equipment, not including ponton material. Under (b) would be included such equipment as can not be readily obtained on short notice in the commercial markets; e. g., ponton material, the minia-

ture intrenching tools issued to the infantry, tools for military mining and other siege matériel, wagon and pack equipment (army standard), portable map reproduction outfits, etc., etc. An ample reserve of the second class (b) should be kept on hand at all times, as there would be a considerable delay in obtaining this in sufficient quantity on short notice, in the event of war, on account of its "special" nature. An additional source of supply, besides arsenals and home markets, is the immediate theater of war whose resources should be utilized to the fullest possible extent. In a thickly settled civilized country great quantities of supplies would be locally available. In a thinly settled or uncivilized country it would usually be unwise to count on anything except the materials of nature.

In equipping the engineer troops to take the field, the chief engineer officer of the army will consider the probable nature of the operations and course of the campaign, the local theater and its resources. He would then consult the lists of tools and equipment available at the home depots or bases of supply, and select such as seemed best calculated to meet the most probable needs. Possible needs should be provided for by having tools on hand at the base or at more advanced points, so that they can readily be forwarded when needed. The weight and volume of the equipment should be such that it can be carried in the regular authorized wagons of the engineers. It should include only such *material* (as distinguished from tools), the need for which can be foreseen, as there is no reasonable hope of obtaining in sufficient quantity in the theater of operations. It is to be remarked that it will usually be better to over-equip rather than under-equip the engineer troops at the opening of the campaign. Any excess of equipment over the immediate needs can always be left behind on the line of communications, and brought up again when required. Additional supplies which may be required as the campaign progresses can be sent up as needed.

Tools for pioneer work, like the structures built therewith, should be of the simplest character. A man trained to their use can do excellent work with the simplest tools. The standard pioneer tools and supplies for a company should include picks, mattocks, crowbars, axes, adzes, shovels (mostly D hdl., r. p.), hammers (asstd.), machetes, hatchets, saws, two-man saws, wire cutters and pliers, a few light jacks, augers (large sizes), a few short rock drills, canthooks or peavies, pulley blocks (up to four sheaves), a couple of small chain blocks, lanterns, marline, rope for lashing and tackle (⅜

good size for lashing, ¾ to 1 good for tackle, nothing larger), wire (10 to 14 AWG—a copious supply), spikes and nails, various sizes; flat and round iron for straps, bolts, driftbolts, dogs, etc., a little tool steel, some tapped nuts of same sizes as round iron, and washers (wrought), machine and black oil and oil for lanterns, one or two light portable forges and anvils with a little coal, a couple of grindstones, leather, etc., etc. In addition to these general tools and supplies, there should be several small and compact carpenter outfits, a simple blacksmith's, machinist's and horseshoeing outfit, a farrier's kit, saddlers' and shoemakers' equipments, sketching instruments (to be carried on persons of sketchers), a couple of light transits (to be used also as levels), stadia rods (used also as level rods), a number of metallic tapes, 50 and 100 feet; a simple plumbing kit, twine, tracing tape (for laying out intrenchments), a demolition outfit (40 per cent dynamite is the handiest explosive), a camera and simple photographic supplies, a simple drafting and map reproduction outfit, a field desk and portable typewriter for the company office, etc. The entire equipment, while ordinarily carried in the wagons, should be such that it can be carried on pack mules, all chests and boxes being of a suitable size. There should be an ample supply of the more common tools (especially picks, axes, mattocks and shovels), so that when occasion arises the labor of other troops or of civilians can be utilized. In selecting the special outfits the company artisans should be consulted.

With this simple equipment an engineer company can perform all the operations which will ordinarily be required of it, utilizing such materials as may be found at hand.

Beyond this it does not seem advisable to recommend any standard equipment, except the regular ponton bridge equipage. There is a large number of special tasks for which special equipment may be used to advantage. Such equipment should be obtained locally or sent forward from the base as it is needed. Some of it may be taken with the company, or in the division train if transportation is available and the need can be foreseen. Plows and scrapers are most useful for road work, intrenching, and many other tasks. In a civilized country they can be obtained in almost any locality. For extensive railroad work the necessary special tools must, of course, be provided. They will generally be found wherever there is a railroad. For simple railroad repairs a few claw and tamping bars, track jacks, spiking hammers, wrenches, hack-saws, track

drills and a "jim-crow" rail bender will be sufficient. Traction and hoisting engines and the commercial types of trenching machines may be used to advanage in fortification work and almost any heavy construction. They can not, of course, accompany the engineers in their more rapid movements, but for big jobs in one locality they may be sent forward when needed.

The battalion headquarters may carry some equipment in addition to that of the companies. Extensive survey work and map reproduction should preferably be under the immediate charge of the battalion commander; the equipment therefor, especially lithographic (zincographic) outfits being with battalion headquarters, operated by special skilled men.

For rapid communication between the separated portions of an engineer command, engaged at section work at various localities, a small micro-telephone outfit and a few bicycles should form part of the company equipment. All engineer officers should be mounted.

The bridging work of the engineers is of great importance, constituting one of their chief duties. The types of bridges are so numerous that no attempt will be made to describe them herein. Considerable skill on the part of the officers is required to adapt the bridge to the site and materials available, to produce the quickest results. The construction of a bridge is an interesting task, and there is a strong tendency to put in a bridge when a reconnaissance a little up or downstream would have revealed a practicable ford near at hand; or to build a bridge at some site where an improved ford with suitable approaches would have served the purpose and have consumed far less time. Where the depth of water and the span are considerable, some form of floating bridge, preferably, of course, the regular equipage, will usually best meet the situation. A site should be selected where the banks are not too high nor steep, to save labor on approaches. With a moderate depth of water, especially if the banks are quite high and steep, the simple two-legged trestle, frame or pile, with spans of from 10 to 15 feet will answer. If the depth is moderate and the bottom firm, the framed trestle will be quicker. With a soft bottom and water over 5 feet deep, piles may be more suitable. A hasty pile-driver may be improvised with a heavy block of wood raised by tackle and guided by a spider and a few upright poles when nothing better is available, and many other practicable devices will suggest themselves to the resourceful pioneer. For

deep and narrow chasms some form of truss or girder, or, when the nature of the sides permits, a spar bridge (which in double lock is practicable up to 50-foot span) will be suitable. For wider chasms (over 50 feet) the suspension bridge is the best type for field construction. But such sites should be carefully investigated to determine whether a road down to and up from the stream, with a small bridge or a ford, could not be constructed in less time. The essential parts of a suspension bridge are the cables and towers, and where the necessity for bridges of this type can be foreseen steel wire cables with an ample supply of fittings and adjustable suspension rods, or hangers, should be provided. On account of the difficulty of bending heavy cables the size, for field work, should usually be limited to 1 inch diameter, multiple cables being used where necessary. In many situations a combination of types may meet the requirements better than any one singly. Such combinations might include frame and pile trestles, trestles and truss spans, or a combination of trestles with floating supports, the last being a very common type. The construction of hasty bridges may often be greatly expedited by having with the engineer train several wagon loads of floor chess, in a country where suitable material for decking is difficult to obtain in sufficient quantity. Lacking chess or plank, the decking will usually be of poles covered with brush, leaves, and earth or gravel. Such a deck can be constructed quite rapidly when material is available, but is apt to be exceedingly heavy. The roadway of a hasty bridge should be no wider than absolutely necessary, both because of the extra weight, and the extra time required for construction. A 12-foot roadway is unnecessary for the passage of foot troops; and even for wagons, 8 feet in the clear will usually answer, although of course 12 feet is more convenient. A wagon bridge should always have side rails (wheel guards), and these are usually required in any case to secure the decking. A hand rail will seldom be necessary.

The regular ponton equipage is by far the most important of the engineers' bridging matériel. This equipage, as a rule, is not regularly assigned to the companies, but is held concentrated as a central reserve to be used where needed. It should always be released as soon as practicable, to save wear on the equipage and render it again available in the advance, a bridge of some other type and of local material being placed alongside for this purpose as soon as possible. For wide spans and deep water the

ponton equipage furnishes indeed the only practicable type of hasty military bridge, unless the time and material (as well as plant) for a great suspension bridge, or a truss bridge on heavy crib or permanent piers, is available, which will seldom be the case. Moreover, for the passage of a large force several crossings will be desired. In view of these large demands which can be met in no other way, it would be unwise to fritter away the ponton equipage in a number of small detachments. Portions assigned to divisions (of troops) remain under the control of the chief engineer thereof, and are operated under his direction by engineer troops specially assigned to the task. There being no troops permanently attached to the ponton equipage, each division thereof should be provided with a small, permanent, enlisted personnel, including the teamsters, a cook and a few men for care and maintenance. All ride on the wagons.

The adaptability of the ponton equipage is very great. In connection with trestles, a few of which are provided with the equipage, it will be the quickest form of bridge in most situations. In the United States Service two classes of matériel are provided, known as the heavy and light, with wooden and canvas floats, respectively. This equipage has amply met our requirements in the past and has suffered little alteration in fifty years. Meantime, all the foreign armies have replaced the heavy wooden pontons with lighter and less bulky floats of metal—even the French and Austrians, from whom we copied our present heavy equipage. They have never looked with especial favor on the canvas boat, except in Russia, where it originated. Such experiments, few in number, as have been conducted with metal floats in our service, have not induced us to make a change, and many officers believe that the canvas floats will meet all requirements—as they did so well in Sherman's Atlanta Campaign. Nevertheless, in view of the universal adoption of the metal float in foreign services, it would seem that further experiments should be undertaken. The wooden boats are very heavy and bulky to transport.

In the attack of a river line, and in other situations, occasions frequently arise for the rapid passage of foot troops at one or more points. For this purpose a very light bridge only, is required, but the time required to construct such out of local materials, in the absence of the regular equipage, might be so great as to defeat the purpose of the crossing. Recognizing this, the Germans have provided a portable (floating) foot bridge to be attached to engi-

neer troops with small commands. This equipage certainly merits our attention. It would be desirable to have a single equipage available for both purposes; that is to say, for either train or foot-bridges, and it does not appear that such a design would present any insuperable difficulties. The length of the bays would be fixed by that of the balk, but by a division of the floats (into two or more sections capable of being combined for the heavier bridges) and the use of fewer balk in each bay, the linear capacity for foot bridges could be made double or treble that for wagon bridges, or even more if additional floats could be obtained locally. A good form of flooring for foot bridges would be a light woven material on the order of a Venetian blind, to be transported in rolls like carpet. However, as this material would be too light for train bridges, a better plan, perhaps, would be to use the train bridge chess laid longitudinally or diagonally on small sleepers placed across the balk with some form of clip or lashing to hold the chess in place. A floor, 3½ or 4 feet wide, would pass infantry in file at a run, or in twos at double time.

Most of the operations of the mobile army in time of war will necessarily be in the field. Therefore, field training is of vastly greater benefit than garrison training. In fact, during much of his time in garrison a soldier is actually learning nothing useful. Maneuvers should take place annually and occupy a considerable period of time. Outdoor training in garrison should be conducted whenever weather permitted, and marches and outdoor maneuvers should not be dispensed with even in the winter. An armory or drill hall, however large and beautiful, can not take the place of the open field in training a soldier.

A Compilation of Experiences and Conclusions of Eminent Authorities as to the Utility of Outlets in Reducing the Height of Floods, Citing all Opinions Found, Whether For or Against.*

PREPARED AFTER EXTENDED RESEARCH

BY

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(1) Result of the diversion of the Adige into the Castagnaro, Italy.—Elevation of flood level by divided flow and, conversely, a lowering of flood level when waters were restored to one channel.

M. R. C., 1890, page 3106:

About A. D. 1438 the Adige broke its levees and poured its waters south into the Castagnaro and Canal Bianco, which then formed a drainage stream parallel to the Po. In 1545 the break had so increased that two-thirds of the low-water flow of the Adige and three-fourths of the high-water flow, went through it. A low dam was built across the Castagnaro to check the flow into it, and both rivers raised their beds. In 1678 a new dam was built, as the old one was then buried in the deposit. The bed still rose. In 1791 a masonry dam, 39 feet high, with many archways through it to allow floods to pass, was built across the Castagnaro. The bed continued to rise, and the floods on the Adige were so high that in 1838 the Castagnaro was permanently closed. In the six years following the closure the floods in the Adige fell, and the more markedly the nearer the point considered was to the Castagnaro.

(2) Extracts from work of Frizi, an eminent Italian engineer, published in 1762—

Of the Rhine, he says:

The great multiplicity of channels, though productive of very

*Being appendix of article entitled "The Use of Outlets for Reducing Flood Heights," by J. A. Ockerson, printed in No. 35 PROFESSIONAL MEMOIRS. (Prepared in August, 1913.)

great advantages to the navigation and commerce of Holland, draws after it very fatal consequences. The waters divided into so many branches lost the rapidity and strength which are required to sustain and push forward those heterogeneous substances which they transport. The constant rising of the bottom renders the drainage of the waters from the fields more difficult, increases the expense of the necessary embankments, and always augments the damage which these extensive lowlands suffer when the dikes break and threaten the whole country in ruin.

To remedy that trouble it was proposed in 1754 to provide still more channels and sluiceways. Gennette, however, maintained that this would not diminish the height of floods; that all of the several channels should be abandoned save one through which the waters should be carried to the sea by the shortest practicable route, and his advice was followed with satisfactory results.

The effect of adding to or subtracting from the volume of a river is described as follows:

We read in the collection of observations for 1728 that, having made the experiment of placing a mark (gage) in the Panaro (river) and of letting in and afterwards withdrawing the waters of the great drain of Burana, they observed in the Panaro no sensible rise in the first instance, nor any visible decrease in the second. These three facts have been particularly attested by Eustice Manfredi, whose testimony is worth that of all others. No objections can be made to these facts, for it can not be said that the quantity of water in the affluent bore no sensible proportion to that of the recipient stream, nor that the sections of the recipient were not effective; neither can the variability of these very sections be attributed to any other cause than an increase of velocity in the united waters, proportioned to the increased quantity of water itself.

. . . In diverting from the principal channel a considerable volume of water, that which is left behind is not visibly diminished either in height or in breadth.

Other early observations on the Po confirm this principle.

It is a hydrostatic paradox, commonly taught by Italian authors, and uniformly confirmed by experience, that you do not diminish the height of the waters in great floods by lessening the quantity of water.

Father Castelli, in the Thirteenth Corollary of his first book on Running Waters, disapproved of the division formally made of the Po at Buondens and which was afterwards abandoned. (In 1638.)

Guglielmini confirmed the opinion as to the little utility of spillways for reducing flood heights in streams.

Eustice Manfredi proved the futility and danger of spillways proposed for the right bank of the Secchio River. Experience, in like manner, demonstrated the futility of a cut made in the right bank of the Arno to prevent the flooding of the city of Pisa.

This cut was made in 1740, but no perceptible diminution of flood height at Pisa occurred. The cut was made anew in 1761, during a great flood, but the waters continued to rise so high that the people "could not be persuaded that the cut had been made." In fact, the greatest flood within the memory of man was recorded.

Cornelius Meyer, a celebrated Dutch engineer, disapproved of this method of dissipating the flood, and instead proposed confining the water to a single channel with ample embankments to carry the flood waters safely to the sea.

The canal made, by order of the Emperor Nerva, to draw off the superfluous waters of the Tiber at the times of its great freshets, did not contribute in the smallest degree to prevent inundations.

One may also inspect the discourse of the celebrated Lorgna on the inundations of the Adige, which sufficiently proves that all the derivations made in that river have only produced a heightening of its bed, and thereby rendered its floods more dangerous.

The following rules, as set forth by Guglielmini, govern the regimen of alluvial streams:

The greater the ordinary body of water in a river, the less will be the slope of its bed.

The greater the quantity of water which a river carries, the less will be its fall; and the greater the force of the stream, the less will be the slope of the bed.

These two rules resolve themselves into a single rule, viz, That the slope of the bottom in rivers will diminish in the same proportion in which the body of water is increased.

(3) The Mississippi and Ohio Rivers, by Charles Ellet, Jr., 1853:

There is in fact no point on the coast above New Orleans at which a sufficient portion of the high water burden of the river can be withdrawn to afford essential relief to the population there, without causing injury to some other community and the destruction of some other interests.

The mode almost universally recommended for obtaining relief

from overflows of the Mississippi is that of creating artificial outlets to draw the surplus water from the river and discharge it through new channels into the Gulf of Mexico.

The earliest suggestion of this plan which has been seen by the writer is to be found in Darby's account of Louisiana, published in 1816. . . .

But it is very certain that the preference almost universally given to this method is attributable mainly to the tempting facility of execution which it offers. . . . The least knowledge of the physical formation of the lower portions of the Delta is sufficient to satisfy every mind that this plan is perfectly feasible and may be executed there with great promptness and efficiency. . . .

But this plan, simple, easy of execution, and certain as it is to effect the purpose, when applied to a limited extent on the lower portions of the river is obnoxious to some well founded objections, and has encountered much opposition from unfounded apprehensions.

He then cites the objections: First. Difficulty of wasting sufficient volume without deluging adjacent country, which he regards as a "solid objection."

Second. Great deposits in the lakes which will impair navigation and finally convert them into swamps.

Third. "Apprehensions seriously entertained and forcibly expressed by engineers of great intelligence and respectability" that the diversion of water from the main river will cause a deposit and reduction of section below the point of diversion which will reduce its capacity and destroy the anticipated relief of the spillway, and no reduction of flood heights will be realized.

Fourth. There is danger of diverting the entire stream through these shorter routes and steeper slopes to the sea, destroying the present channel and leaving New Orleans without water communication.

Mr. Ellet does not agree with the third objection "though the principle which looks for the accommodation of the size of a river's channel to the volume of water which passes through it, along a given slope, is fully admitted." "Certainly there are in a channel formed in an alluvial soil reciprocal relations between the depth, breadth, slope, velocity and discharge."

He says it does not follow that the capacity of the main stream will be diminished by spillways "for the reason that it requires more force to create than is required to maintain the channel after its formation. . . ."

The water which injures the country is not that which flows

down between the natural banks or the levees, but that which covers the fields when the levees break.

We may surely discharge this portion through artificial openings leading to the sea without affecting the area below; for it does not now and never did flow through the channel and can have therefore no influence whatever on its condition. . . . To the extent then . . . of discharging . . . the crevasse water, we may employ artificial outlets without the least apprehension that the present area of the river (cross-section) will be diminished,

and this expedient may be extended

until we approach the unknown limit which represents the difference between the volume needed to create and that needed to maintain the channel. . . .

These objections to the use of outlets to a limited extent are not tenable. It is therefore proposed to resort to high water vents so far as is necessary to obtain prompt though limited relief from pressing distress and impending calamity, but not to rely on this expedient exclusively, or even look to it for full protection or permanent security. . . .

These great purposes will be aided but not accomplished by outlets, which, therefore, are now only recommended for local relief and limited application.

He recommends an outlet at Lake Borgne with an opening of 5,000 feet in width at the river and estimated discharge of 210,000 s. f., which would "lower the flood 2½ to 3 feet;" also outlet at Bonnet Carre to Pontchartrain.

He also recommends the enlargement and maintenance of both the Atchafalaya and Plaquemine. The latter to carry 120,000 s. f. from its then capacity of 30,000 s. f.

The Atchafalaya at its source draws from the Mississippi in very high floods about 85,000 s. f., or 8 per cent of the discharge of the main channel. . . .

If we allow for the increased duty of the Atchafalaya 150,000 s. f., we must assume that the capacity of the channel . . . must be thrice as great as it is now.

If the depth can not be materially increased then area must be obtained by increasing the width "about 1,600 feet, or about 1,000 feet greater than it is now." This means the excavation of 1,000 feet wide, 50 feet deep and 80 miles long.

With regard to an outlet into Lake Pontchartrain at Bonnet Carre, Mr. Ellet says:

Next to the enlargement of the Plaquemine, the most efficient

outlet that can be made at reasonable cost and in reasonable time . . . is from Bonnet Carre bend to Lake Pontchartrain. This expedient has been frequently proposed and elaborately discussed by others. . . .

The leading . . . argument in favor of this plan is unquestionably the great rapidity and ease with which it can be accomplished.

The objections are—

1. "That the point where the outlet is proposed to be made is too near the Gulf to afford relief to any great extent of river coast."
2. That the deposits from the river will "ultimately destroy navigation on Lake Pontchartrain, which must always be of great value to the commercial life of New Orleans."
3. That the water will raise the surface of the lake so as to inundate the swamps on its coast and in the rear of New Orleans, rendering backwater levees and pumps necessary to provide for drainage.

The objections . . . enumerated are so serious that a resort to this measure, so simple and so certain to produce prompt but limited results, can not be recommended here.—At least . . . as a permanent improvement and reliable plan, though it may ultimately be adopted should events arise to justify it, as a temporary expedient.

The amount proposed to be carried by this spillway is 100,000 s. f. and sediment deposited in the lake by a flood lasting 60 days is 288,000,000 cu. ft.

Bayou Lafourche, given as discharging 10,200 s. f. at high water. Levees 10 to 20 inches wide on top.

As to cut-offs—

It is . . . recommended that prompt measures be taken to prohibit all attempts to effect or encourage cut-offs in future, as well as to guard against their occurrence from accidental causes.

As to volume of confined flood—

To afford the needed protection, we must provide means adequate to give vent to or to resist an increased discharge of at least 700,000 s. f., or about seven-tenths of the present high water discharge of the river.

As for Louisiana—

for this State there is no alternative. . . . She must build levees without hesitation or delay.

Proposes a line of "guard levees" from Red River down, back

of existing line, crown to be 6 feet above highest known floods, at an estimated cost of $12,000 per mile.

Regarding reservoirs.—Concludes that to reduce flood 1 foot would require storage of 20,000,000,000 cu. ft. per week, or for a sixty-day flood, reservoir 630 sq. miles, 11 feet deep, would be required.

Inquiry by Louisiana Legislature discloses the fact that in 1800 there were 25 to 30 bayous or outlets from 50 to 60 feet wide between Plaquemine and Red River on right bank.

(4) Maj. J. G. Barnard, Corps of Engineers, U. S. A., 1859, from letters originally published in *New Orleans Picayune*, revised and republished in *De Bows Review*, October, 1859.

There is but one protection to Louisiana (from floods) and that is levees; outlets or natural vents of any kind may be discussed—adopted by State authorities—perhaps attempted. If so, they will certainly deluge the unfortunate district through which their discharge is carried, while they utterly fail to relieve the river, producing on the other hand deposits in its bed, which they will eventually raise, and with it the surface.

In 1850, the bed of the river below the Bonnet Carre crevasse was measured by Professor Forshey and others, who found that immediately below the crevasse it was obstructed by a shoal which diminished the mean depth about 12 feet, and the area of section 75,613 square feet, as compared with sections above the crevasse.

Professor Forshey adds: "Soundings made in the high water of 1851 proved that the great sand bar thrown across the channel of the river below the crevasse (of 1850) by the weakness of its transporting forces, was carried away again when the river was confined by the new levee, and rose high enough to assume its greatest velocity. With those accustomed to the study of hydrodynamics, the above facts furnish a fortunate illustration of the effect of outlets."

General Barnard continues:

Similar effects have been observed to be produced by other crevasses (witness the extraordinary deposits in front of the New Orleans city wharves in 1858, caused by the Bell crevasse just above the city limits), and it is doubtless an effect attending all crevasses and will attend all outlets, should the folly of making or attempting to make them be attempted. . . .

No lateral vent exists—none can be made by man—which will carry off these one, two or three hundred thousand cubic feet, which it is wished to dispose of, so safely as the river channel itself, aided by sufficient levees.

The waters of Bonnet Carre crevasse maintained, according to observations at Fort Pike, a permanent level at this lowest point of

the lake (Pontchartrain) of about 18 inches. How perfectly inadmissible such a permanent elevation of back waters would be to the immense number of valuable plantations which drain into Lakes Pontchartrain and Maurepas—how absolutely ruinous it would be to New Orleans and its suburbs—yet this is by far the most favorable case of the outlet system.

The greater the volume of water the less fall or head is required to move it with a given velocity and therefore the less surface elevation.—It is implied also that the river molds and changes its bed to adapt it to the volume to be discharged, which is true of such rivers as the Mississippi.

Given a certain channel of unvariable form carrying a given volume of water, and the proposition that an additional volume of water thrown into it does not raise the surface bed, is not only a paradox but an absurdity.

But let the channel be a river bed whose sides and bottom are formed by the conflicting agencies of deposition and abrasion, and then the paradox becomes true, not as an immediate but as the ultimate result.

(5) Humphreys and Abbot Report, 1861, page 428, concludes a long discussion of Outlets as follows—

Enough has been said to demonstrate, with all the certainty of which the subject is capable, the disastrous consequences that must follow the resort to this means of protection.

(6) The Board of Commissioners appointed "to investigate and report a permanent plan for the reclamation of the Alluvial Basin of the Mississippi River subject to inundation," provided by Act of Congress June, 1874; Report Chief of Engineers, 1875, page 539. With regard to cut-offs, the Board states: "It is therefore the emphatic opinion of this Commission that in every case they should be prevented." (Page 540.) As to diversion of tributaries, the Board states that it is impracticable and would be injurious to main stream to eliminate a portion of the flow. (Page 542.)

Reservoirs: "No relief . . . simply because there are no available sites for reservoirs sufficiently large to produce the desired effect." (Page 540.)

Outlets: "This Commission is forced unwillingly to the conclusion, that no assistance in reclaiming the alluvial region from overflow can judiciously be anticipated from artificial outlets. They are correct in theory, but no advantageous sites for their construction exist." (Page 548.)

Cut-offs regarded as very pernicious. Outlets find no useful application on the Mississippi River. (Page 564.)

Estimate for Lake Borgne outlet of 150,000 second-feet, $1,122,311 (page 629). Board estimates rise of back water in Pontchartrain of 4 feet, if spillway to carry off peak of floods is installed at Bonnet Carre.

Board recommends a levee system extending from the head of the alluvial basin to the Gulf and including the valleys of the tributaries.

Further recommends that Plaquemine be opened and Lafourche and Atchafalaya be maintained; a double line of levees below Red River suggested. (Pages 560-564.)

(7) Mississippi River Commission in Report dated February, 1880 (Report Chief of Engineers, U. S. A., 1881, page 2725).

It has been supposed by many persons that because the immediate effect of a crevasse during a flood is the reduction of height of the river's surface in the vicinity of the crevasse and below it; lateral outlets, either natural or artificial, by which the flood waters of the river are drawn off and conveyed through a shorter route to the sea, tend to prevent the recurrence of destructive floods by supplying additional avenues for their escape. This method would undoubtedly be effective if the flood waters of the Mississippi were not highly charged with sedimentary matter, which are held in suspension in the water by the current. . . .

The fact seems to be established that when the current is checked in its natural flow during floods, a deposit of sediment will occur. Shoals are found in the river below crevasses, which it is difficult to refer to any other cause than to loss of current velocity which takes place below the crevasse. As a portion of the volume of the river is drawn off by the crevasse when it is first made, it is impossible that the current below the crevasse should then be as rapid as it was before its occurrence. Being less rapid, it is unable to sustain the whole quantity of matter held in suspension by the more rapid current above the outlet, and consequently its surplus sediment falls to the bottom below the crevasse. This deposition continues until the size of the river below the crevasse has been so reduced by the shoaling that the current is again restored through the short distance in which the bottom of the river has been thus raised and the channel diminished. . . .

It is a well established law of hydraulics that the ratio of frictional resistance per unit of volume increases if the sectional area be diminished. . . . Whenever such streams flow through alluvial deposits, other conditions being the same, the slope is least where the volume is greatest; and, conversely, the slope is found to be invariably increased as the volume is diminished. . . .

We have the evidence of this relation between slope and volume presented in the phenomena of silt-bearing streams all over the world. . . . As the system of improvement proposed by the Commission is based upon a conservation of the flood waters of the river and their concentration into one channel . . . it would seem scarcely necessary further to consider a system based upon theories and arguments so diametrically opposed to it as the outlet system is thus shown to be. . . . Were the high river confined within such limits of width as theoretical investigation should show to be compatible with an efficient discharge section, and the banks protected from erosion, the scouring power of the stream being concentrated over a moderate area, would soon form a deep channel and the resulting deep and narrow section would be far more efficient than the wide and shallow one of the unregulated river. Owing to decrease in frictional resistance, the volume of flood waters would be passed off more readily and rapidly, the mean velocity of the current would be increased and the flood line would be lowered. (C. of E., 1881, p. 2750.)

C. of E., 1881, p. 2775: The Committee on Outlets and Levees (Harrod and Suter), accounting for a relative difference of 5 feet in elevation of two floods at points 160 miles apart state,

We are therefore of the opinion that the bed is raised and the effective section impaired throughout that part of the river under discussion, as the results of outlets and the abandonment of levees. . . . (1) That bayous and overflows afford no permanent or uniform relief, but produce changes of regimen detrimental to navigation, and cause destructive floods.

Depression of Bed Due to Levees.—M. R. C., 1881, page 128: Levee building on Red River below Shreveport began about 1860—bed of river has been depressed and "lands have not been inundated for years."

M. R. C. report, 1883, p. 16: Effect of crevasse or outlets on bed of stream below shown by surveys above and below four different crevasses of 1882, viz, Malones, 2,200 square feet; Riverton, 11,000; Bolivar, 8,400; Mound Place, 23,800; in other words, a marked reduction of section in each case shown by comparing averages of several cross-sections above and below breaks.

Results were also obtained at and near Bonnet Carre by comparing cross-sections with crevasse flowing and after closure shows scour after closure amounting to 12 per cent of low water area. The crevasse had been open for eight years, closed in 1882 by State, R. R., and M. R. C., at a cost of $90,000.

M. R. C., Annual Report, 1884, p. 14:

As the direction of the surface slope affects the direction of the

thread of motion of the stream to all depths, so in case of both cut-off and crevasse the stream is turned, involving a loss of power in the impact against the bank; continuity of motion is broken, velocity is reduced and a source of bar formation is introduced. In case of the crevasse, by loss of volume, loss of velocity and ultimately of section, is also incurred.

Page 15:

Cubitts Gap enlarged until reaching 34 per cent of capacity of main stream. Produced deterioration of river amounting to 4 feet, which continues.

Page 16:

The bed of the river has adjusted itself to the annually recurring demands of the discharge by increasing its depth where they are greatest and diminishing it where the discharge is dissipated by successive outlets.

A. R., M. R. C., 1884, p. 18:

The dimensions of the river are of its own creation; that a large volume collected in one bed is discharged with less friction, other things being equal, therefore with less slope, and more beneficent results to navigation and riparian interests; while subdivision of volume, on the contrary, meets proportionately more resistance in each minor channel and therefore requires more slope.

It follows from this law that every step in the improvement of the river . . . should be in the direction of the assembling of its many outlets into one channel, while subdivisions in passes or dispersion through crevasses is a grave error.

A. R., M. R. C., 1885, p. 2865:

The outlet system, by which a portion of the flood waters of the river would be drawn off and conveyed through shorter routes to the Gulf, being one of diffusion and waste, and having very little in the way of either theory, experience or observation to recommend it, was unanimously rejected by the Commission (in 1880).

Page 2872:

The Commission is distinctly committed to the idea of closing all outlets . . . both low water outlets and breaks in the levees and it has consistently opposed the fallacy known as the "outlet system," because it stands in sharp antagonism to those principles and to all fundamental laws of hydraulics.

1890 Vicksburg, Miss., River Improvement and Levee Convention. A. R., M. R. C., p. 3271:

We unhesitatingly express our condemnation of all theories for

the regulation of the Mississippi River and controlling its floods by the agency of outlets; it is our conviction that such a system would prove destructive of all of the interests which are sought to be conserved and improved in connection with the treatment of the Mississippi River. (Signed by three officers, U. S. Engineer Corps; two members M. R. C.; twelve levee engineers, U. S. and State.)

M. R. C., Rept. 1890, p. 3105:

Gen. C. B. Comstock—

Outlets have often been proposed as a method of reducing flood heights on the Mississippi. The immediate results on flood heights are so evident and so beneficial when a large crevasse is formed, the good results of an opening far larger than ever occurs naturally seems so immediate and apparent that it is not strange that many persons look on them as the true remedy for great floods. . . .

"Sedimentary rivers flowing in their own alluvion" take larger slopes the smaller they are (cites several examples). "Hence, if at Lake Borgne or elsewhere in its delta the Mississippi was divided into two rivers, since each would be smaller than the present river, the two new rivers would go to work to obtain the new and steeper slopes suited to dimensions smaller than those of the original river, and hence would build up their beds. This process would only cease when the steeper slopes needed by each were obtained. Since both rivers would have one end at the Gulf and have steeper slopes up to their point of divergence than the main river now has, the flood surface of the river at that point would be higher than now. . . . It may be concluded that the reduction, by any large amount, of the flow of the Mississippi at Lake Borgne (amount under consideration 400,000 s. f.) will be ultimately followed by a rise in the flood heights at that place and a shoaling of the river below and at its mouth. . . . No important outlet at Lake Borgne should either be undertaken or permitted, because it would for a time accelerate velocities above it, increase caving and destruction of levees; the flood heights at Lake Borgne and New Orleans would after a time be greater than they are now.

M. R. C. Rept. 1891, page 3684.—Captain Millis states about Ames Crevasse:

The crevasse undoubtedly produced considerable disturbance in the current at the lower discharge section, as is indicated by the formation of a bar in the river near that point where there was formerly deep water.

(8) Investigation of effects of crevasses on bed of stream. Capt. C. F. Powell, Corps of Engineers, U. S. A., M. R. C. Rep. 1891, page 3467.

Morganza Crevasse, 1883-1889:

On the whole, and from a consideration of all the measurements, the only conclusion that seems warranted by the record for Morganza is that the influence of river stage greatly predominates over the outlet effect, and masks the latter so closely that neither its amount or its direction can be stated.

Regarding the examination of the Crevasses of 1882 heretofore cited, Captain Powell says:

The somewhat decided average fills at sections below the crevasses named, as shown by the surveys of 1882, have been prominently cited as positively proving that outlets do raise the river bed. . . . The sections are not comparable for the purpose desired, there being no way to eliminate effects of river stage or condition.

Bonnet Carre Crevasse, 1882-1884:

Measurement of sections from the crevasse drawn in 1882 after crevasse had been open eight and one-half years, again after it had been closed one and two years, respectively, survey of 1883, shows an average increase of section below the crevasse of 3,343 square feet, and a year later an additional amount of 16,041 square feet.

Captain Powell says:

The increase of mean sectional area from 1882 to 1884 . . . apparently indicates a scouring from a closed crevasse over that of an open one, which inference is confirmed by the measurement of the intermediate year.

With reference to outlet effect of the Atchafalaya, Captain Powell says:

While it may not be safe to base any general law on the effects of the Atchafalaya outlet, it can be said that the filling of the main river bed below is not fully and satisfactorily referred to other cause.

His general conclusions state:

For outlets at and below Red River (from where they can be led directly to the Gulf) that as far as the record goes, the large all-stage outlet was found to cause a measurable and remaining fill in the main river below, but not the smaller and high-stage crevasse. When the latter ceases to flow, the effect of the river is by scour to return to its normal section of bed and the action of 10 to 20 per cent of the whole river for about one-fourth of the time ought not to decidedly influence it for the whole time.

(The conclusions are not borne out by the results at either Bonnet Carre or Morganza, and instead of "10 or 20 per cent," spillways for effective flood relief must carry fully 25 per cent of the main volume as shown by Nita Crevasse, which flowed 400,000 second feet and lowered stage at Carrollton 1 foot as deduced by Captain Kingman. Colonel Suter makes it 2.1 feet.)

(9) Spillways below Red River for the purpose of lowering flood heights. Capt. John Millis. 1894.

To justify the sacrifice of the investment which this excess of height and strength (of levees) represents, a plan to definitely restrict the height of the water to a lower level than that which the present project for a levee system contemplates must present the advantages of economy, as compared with the cost of completing the work on the original plan, of time required to complete so far as to be effective, and of diminished risks involved during the process of construction. Above all, such new plan must present the recommendation of certainty of results. . . .

That such relief is afforded immediately and effectively by breaks in the levees at high stages of the river is well known to everybody who has observed a flood below Red River; and that no appreciable relief is produced, so far as past experience and observations go, by an opening of considerable depth and limited width which remains constantly open, is equally well established.

"A very long opening with its bottom sill at or near bank level has been frequently suggested." . . . The construction of such an opening would have to be regarded as an experiment in many respects and the effects could not be predicted with entire certainty. . . . It remains to determine whether the amount of the risk now uninsured and the degree of relief and protection which cutting a levee at high water affords, will justify the outlay necessary to prepare beforehand one or more places where cuts may be made without such injury to other interests as to condemn the plan as a measure of security. . . .

It seems more nearly correct to regard the effect of a break as resembling the tapping of a large reservoir, rather than taking off the top of the flood in the sense of diverting a flowing stream.

Captain Millis then suggests three possible sites for spillways: Bonnet Carre, Kenner, and Lake Borgne, and recommends the construction of a spillway at Kenner, 2,500 feet wide, with capacity of 200,000 second-feet at maximum stage estimated cost, $280,625.

Estimate for similar spillway at Lake Borgne, $194,900.

In conclusion, Captain Millis says:

I believe that the providing of some measure of relief, to be

available in case of emergency for preventing a repetition of the disasters of the last four years, is of such immediate importance as to demand serious consideration at once. I also believe that whatever may be the standard of height and strength to which the levee system below Red River is finally completed, some such provision as that herein proposed for relieving the river in the event of an extraordinary flood will be a judicious and economical measure.

(For further details see pamphlet M. R. C. print July, 1894, entitled "Restricting Flood Heights, Lower Mississippi River," by Captain John Millis.)

(10) M. R. C., 1893, pages 3560-61. Referring to levees as the only protection from floods—

To any other system for the protection of these alluvial lands there are objections, both theoretical and practical, which can not be overcome. Those of a theoretical character are connected with the hydraulic law that a reduction of flood discharge in a silt-formed channel will reduce its capacity below the point of outlet. This observation has been repeated in the many local comparative surveys of the river bed made by the Commission before and after crevasses, or before and after rebuilding the levee made necessary by a crevasse. The difficulties of a practical nature consist of the necessity of continuing any outlet to the Gulf without return to the river at any intermediate point. Such return would merely transfer the danger from one locality to another lower down. The condition thus imposed upon any outlet involves great cost and danger. Any outlet intended to relieve the parts of the river where flood heights are now most dangerous would, in any case, be very long. To have an appreciable effect it would require an excavated channel of large width and depth. As it would traverse tracts of cultivated and valuable land, the cost of expropriation of a right of way would be very great. The cost of the levees, which would be required to control it and prevent the permanent continuation of disaster and overflow while flowing through regions which will in a few years be placed under the sufficient protection of a levee system, would be very much greater.

When completed there would be two or more rivers, each presenting the same dangers and requiring the same treatment as the present single channel. The cost of efficient outlets would be greater than the cost of completion of the levee system. When constructed they would either silt up, as do many island chutes, or would enlarge and assume the tortuous type of an alluvial stream, with shifting bars and caving banks. Finally, conceived and executed in defiance of physical law, they would fail as a safe, cheap, or efficient method of abating overflow.

Minority Report, M. R. C., 1893, prepared by General Comstock:

signed later by Flad and Whiting; signed by Comstock, Ernst, Flad and Whiting:

Concurring in the inadvisability of an attempt to create new outlets from the Mississippi River which shall be large streams at all stages of the river, we do not wish to be understood as condemning the use in the levees of long waste weirs to take off the top of the flood if it shall be found that at certain places in the lower part of the river the further increase in the flood flow, which will come from raising the levees further up the river, can be controlled in whole or in part by such waste weirs more economically than by higher levees.

(11) Report of Senate Commerce Committee, 1897:

Your Committee can discover no just or adequate relief (from floods) in reservoirs. Neither can your Committee discover, from the evidence or through other sources, any material relief from the outlet system. It is not practicable to relieve the river by means of outlets except below the Red River, 768 miles below Cairo. But these outlets, or others that might be constructed on this reach of the river, could afford no perceptible relief for the river above, where relief is much more called for and needed. There is no disposition to substitute outlets for levee enlargement.

From all the evidence taken and considered by your Committee, it is evident that the basins and bottoms along the Mississippi River exposed to floods of the river can only be protected and preserved from such floods by an ample and complete system of levees from Cairo to the Head of the Passes.

OUTLETS.

Extracts from address and writings of R. S. Taylor, in 1884—

The effect of outlets upon the flood levels and upon the river channel has been the theme of great controversy for a generation past. There is a class of aquatic doctors who regard the Mississippi in every time of flood as sick, whose diagnosis of the case is dropsy, and whose remedy is tapping. Bills have been introduced in Congress, and vigorously pushed, to provide for the making of vast outlets by artificial means. The opponents of such measures have claimed that the effect of such diminution of volume in the river is, to lessen its energy and transporting power, and so cause deposits of sediment, which choke up the channel, increase the flood heights, and thus make the last state of the river worse than its first. These views have been supported by many observed facts, and by what seemed to be unanswerable reasoning. Nevertheless, there has been felt by intelligent students of the question a strong desire for more facts, and for facts based on observations so taken as to afford the highest possible guaranty of their accuracy. Some such facts we are now able to give.

During the great flood of 1882, a number of crevasses occurred below Memphis. Less than three months prior to that time a survey of that part of the river had been completed, so that we know its exact depth, width and cross-section at those points as they were before the flood. After the flood a resurvey was ordered to be made in the locality of four of those crevasses, for the express purpose of ascertaining what change, if any, they had produced, in the river channel. The results were as follows: At Malones Landing, where the upper and least crevasses of the four occurred, there was a shoaling of the channel below the crevasse amounting to 4 per cent of its cross-section; at Riverton there was a shoaling of 14 per cent; at Bolivar, 11 per cent; and at Mound Place, 24 per cent.

At Bonnet Carre, a few miles above New Orleans, a crevasse occurred a number of years ago through which a large volume of water escaped into Lake Pontchartrain. It remained open until the autumn of 1882, when it was closed. We had a survey of the river adjacent to this crevasse, made before its closure, and have just completed another made since. A comparison of the two shows that since the closure of the crevasse the channel space in the river below it has increased about 12 per cent. We have, therefore, in four cases, a filling up of the river bed following an outflow through a crevasse, and in one, a scouring out of the bed following the closure of a crevasse and the stoppage of its discharge. I am not now arguing the outlet question, and will not take time to comment on the significance of these facts; but that they weigh more than a foolscap quire of conjecture will be readily admitted.

All plans for the improvement of the Mississippi River that have been prominently discussed rest upon one of two theories: One, that the remedy for the evils existing consists, essentially, in the concentration of the water of the river and the equalization of its flow; the other, that it consists in the subdivision and diffusion of its volume. The former theory is the one held by the Commission and adopted by Congress, though the other has advocates whose views continue to be urged in Congress and through the press.

There are numerous opportunities to let out of the channel of the Mississippi large portions of its water. The Atchafalaya, which receives its water from the Red River, at a point only 6 miles distant from the junction of that river with the Mississippi, is there several feet lower than the Mississippi. At the present time the water of the Red rarely reaches the Mississippi at all, but, for the most part flows down the Atchafalaya, augmented by a large flow from the Mississippi. It is only when the Red is high and the Mississippi low, that there is any flow from the former to the latter. The difference of level between the Mississippi and the Atchafalaya increases as they descend, so that at Plaquemine the difference is 22 feet, and the distance between them 11 miles. On the east bank, at Bonnet Carre, the difference of level between the river and Lake Pontchartrain is 20 feet, and the distance between them 5

miles. At Lake Borgne the difference of level is 14 feet and the distance 6 miles. So that it is quite possible to make outlets by which the floodwater, or, for that matter, all the water of the Mississippi, could find its way to the Gulf by shorter lines and lower levels than those of the present river. And when men stand appalled at the sight of its devastating floods, the suggestion of quick relief by outlets seems at first sight so practicable and reasonable that it is not strange that it should find favor in many minds.

The reasons which lie against this theory require some thought for their clear apprehension, but rest, as is believed, on well ascertained laws.

If anyone will set a tumblerful of the coffee-colored water of the Mississippi where it will remain undisturbed for twenty-four hours, he will find, at the end of that time, a teaspoonful of mud in the bottom of the tumbler, while the water will be comparatively clear. If he had kept the water in motion all the time he would have found it as turbid as when he took it from the river, and no deposit at the bottom. In this tiny experiment is illustrated the most important law of the Mississippi River's life, viz: That the power of water to sustain and transport sediment depends upon its motion. The exact relation between the velocity of moving water and its silt-carrying power is not ascertained. It varies with the size and specific gravity of the particles. But enough is known to warrant the general statement that every diminution of the velocity of a running stream diminishes its silt-transporting capacity. It is not necessary, in order to produce this result, that there shall be a cessation of motion in the water; a diminution of its rate is sufficient.

The operation of this law is universal and invariable, on the largest scale and on the smallest. The miniature torrent that sweeps down the plowed furrow in a hill-side field scours its bed clean as long as it maintains its headlong velocity. But when it slackens its course at the foot of the slope, it throws down its load of sand and loam in an instant. The delta which forms at the mouth of every sediment-bearing river in the same phenomenon on a larger scale.

The greatest law of the river's life is that it shall be forever laden with a burden which slips from its grasp the instant it loiters by the way. The whole valley is itself the result and the proof of the existence and the operation of this law. There is not a shovelful of earth in all its square miles that has not been dissolved in water, and carried by water to the point where its journey ended for want of velocity to carry it farther. Every grain of sand, every drop of water, every inch of movement, every low stage, every high stage, every flood, is equally obedient to the same great law. The engineer who has learned its meaning well has taken the first step toward a successful solution of the problem of the river's improvement. He who ignores or defies it, put himself in the pathway of forces as irresistible in their operation as the march of a glacier.

This brings us to the practical question. Here are so many cubic miles of water per annum to be conducted to the sea. It is desired to do it in such a manner as to afford the best possible low-water navigation, and the least possible high-water inundation. Shall it be confined in its course to one channel, or shall it be subdivided or allowed to subdivide itself into several?

The excavation of a river channel is work. The transportation of sediment is work. The accomplishment of either involves the expenditure of energy. So that it is but another statement of the same question to say: How will a given quantity of water perform most work—in one concentrated volume, or in subdivisions?

To this inquiry the experience of every person furnishes a ready and true answer. A gentle rain, falling on the earth's surface, leaves its lightest vegetable mold undisturbed. The same quantity of water descending in a water-spout, tears away its solid hills. The water which trickles over a field through thousands of infinitesimal and broken channels, without producing the least abrasion of its surface, would, if concentrated in one volume on the same surface, plow it deep and wide. There is nothing else in nature of which it is so literally and familiarly true that "in union there is strength," as of water. Subdivided finely enough, it is the morning mist; concentrated largely enough, it is nature's master mechanic, the continent-builder of the world.

Taking its whole life together, there is, in the phenomena of the Mississippi River, as elsewhere, a perfect correlation of forces. It does all the work it can all the time. It excavates its bed as deep and wide as it can. It carries out of its channel as much of the sediment mixed with its water as it can. When it drops from its broad shoulders a hundred thousand cubic yards of sand on one bar, as it often does, it is because it can carry it no farther. It never drops a spoonful except for the same reason.

All the conditions of the river are the result of its own forces. It has no antecedent banks, bars, width or depth; all these are its own creation. It made yesterday the things that make it to-day; it is making to-day the things that will make it to-morrow. Its present inadequacy to meet the wants of men is the result of its inadequate work in the past. It overflows its banks because its channel is not capacious enough to contain its flood. To make its channel more capacious requires more work. At low water it stumbles over bars which it lacks strength to displace. To remove them requires more work. It makes its shoal places shoaler by the deposit of sediment which it wants energy to carry farther. More work would do it. So that if the river of the future is to subserve the interests of mankind better than the river of the present, the river of to-day must be induced to do more and better work. This is to be accomplished by the conservation, concentration, and wise direction of all its energies. That this means the conservation, concentration, and wise direction of its volume seems too obvious to need repetition.

Depletion, or subdivision, is a step in exactly the wrong direction.

It is a change toward weakness; not toward strength. An outlet may serve to *lower a flood for the time being,* but the result is a *temporary advantage* gained at *enormous future cost.* The *diminution of volume* produced by the outlet means *diminution of energy.* It means a smaller, weaker river from that point to the sea. And that means a river less able to remove bars, less able to excavate a channel, less able to carry sediment, less able to contain a flood.

It is in the nature of things that each portion of the river, shall, in its turn, bear the burden of the complications above it. The bars that obstruct it come down from above. Hence, there is the clearest necessity that it should, as in the order of nature it does, augment its strength as it descends. *An outlet reverses this order of nature* and of reason. It diminishes the *energy of the river below,* without lightening, in any like degree, the burdens cast upon it by the river above.

At its mouth the Mississippi River has tried the outlet system of it own accord, by subdividing into four distinct channels. From New Orleans down to the widening of the channel, preparatory to its subdivision, it has nowhere less than 100 feet of depth. In its four subordinate channels the depth rarely exceeds 50 feet; and at the mouths of all, except the one improved by the jetties, it is about 16 feet. Suppose the same subdivision had taken place 200 or 300 miles higher up, would not like results have followed in the channels below?

Outlets in the form of *great crevasses* have frequently occurred. They are always *followed by increased deposits in the bed below them.*

Nature has made the channel of the Mississippi of increased capacity for discharge as it descends. The permanent diversion of a substantial portion of its water by outlets will reverse that order, and shrink the channel below them to the size required by the diminished river. This shrinkage will take place first by decrease of depth, as the depositing sediment falls to the bottom, and thus the interests of navigation will be the first to suffer. But the mischief will not stop there. When the river and the outlets shall have adjusted themselves to the new conditions created, they will, all together, be *less able to discharge a great flood than the unimpaired river was alone;* and there will be no place for it to find vent except over the shallow rim of the shrunken channel.

Suffice it to say that *the retention of the river's water in one undivided volume is the cardinal principle* of the plan; and that it is proposed to seek this end by such means as will introduce the fewest artificial conditions into the river's life, and apply most economically and effectually its own forces to do the work of improving its own channel. These consist chiefly in the *closing of chutes and outlets;* the contraction of the channel in places where it is excessively wide, by the creation of new banks from deposited sediment; and the revetment of banks where caving is exceptionally rapid and injurious.

A stream of water loaded to its full carrying capacity with sediment is exceedingly sensitive to change. The particles of sediment swim by means of their friction. Two particles that can barely swim separately will sink if brought in contact, because the contact reduces their aggregate frictional surface. In falling, they will carry down myriads of other particles that can barely swim. So that the least halt in the great procession of particles precipitates part of its constituent membership to the bottom. There they create an obstruction which prolongs the halt; the prolonged halt increases the obstruction, and so on, each condition reacting upon the other. The result is decreased velocity and increased rise in the stream. Just how slight a cause may start this train of causes, it is impossible to tell.

LEVEES.

The great floods of 1882 and 1883 presented in this respect a remarkable contrast. That of '82 had free access into the Yazoo basin through numerous gaps in the levees. Prior to the flood of '83, these were repaired so far as to exclude the latter flood almost entirely. During the flood of '82 the maximum stage at Cairo was 51.87 feet above low water; at Memphis, 35.15; at Vicksburg, 48.75. During the flood of '83 the maximum at Cairo was 51.90; at Memphis, 35.11; at Vicksburg, 43.80. Thus, while the two floods reached almost exactly the same height at Cairo and Memphis, the second fell off 4.95 feet at Vicksburg. The difference is very significant. A reduction of 5 feet in Mississippi flood level means a great deal.

During the years 1881 and 1882, systematic observations were conducted by the Government at several stations on the Mississippi, to determine its depth, cross-section or vertical water area, mean velocity, and discharge, besides some other matters not material here. In these observations may be found some striking illustrations of the tendency of an alluvial stream to obstruct its own path during overflow. At Clayton, Iowa, where general overflow begins at the 14-foot stage, observations taken during the flood of October, 1881, showed that at that stage the velocity of the current, which up to that time had increased as the water rose, suddenly ceased to increase and remained substantially unchanged until the rise reached its maximum of 18 feet, seventeen days later. This is to say, the river rose 4 feet above its banks without any increase of velocity in its current.

Like observations taken at Grafton, Ill., during the flood of April and May of the same year, showed that as the river rose above its ordinary stage, the velocity of its current and the amount of water discharged increased in the usual accelerating ratio until the stage of general overflow was reached. At that point the velocity and discharge ceased to increase, and, during the last 2 feet of rise, both actually diminished. To be exact, on April 25, the stage was 32.17 ft., cross-section 67,545 sq. ft., mean velocity 4.594 ft. per sec., discharge 310,309 cu. ft. per sec.; while on May 6 the stage

was 34.16, cross-section 73,695, velocity 3.859, and discharge 284,405. That is, the river ran with nine inches per second less velocity, and discharged nearly 26,000 cubic feet per second less water at its highest stage than when 2 feet lower.

At Hays Landing, Miss., the gage reading March 20, 1882, was 38.58, cross-section 172,105, mean velocity 5.44, discharge 936,900. Ten days afterward, when the river had passed from general overflow to bank-full stage, the gage reading was 34.14, cross-section 167,396, mean velocity 6.26, discharge 1,049,000. That is, a fall of 4½ feet from the maximum, which brought the river within its banks, increased its velocity eight-tenths of a foot, and its discharge 112,000 cubic feet per second.

In April, 1881, an unusual flood occurred in the Missouri. It originated above Sioux City, and was not augmented by any floods from its tributaries, below. For a distance of 500 miles it covered the valley from bluff to bluff. There was nothing to retard its progress, except such obstructions as it created itself. Nevertheless, its velocity so slackened as it rose above its banks, and its rise so increased as its velocity slackened, that it reached a height at St. Charles 3 feet above its relative height at Sioux City; whereas, the normal course of a flood is to diminish in height above the low-water plane as it flows downward. It was observed, also, that the amount of water discharge was less than that which the river was accustomed to discharge at a height 7 feet below that of the flood and within the banks.

The Lafourche is a bayou heading in the Mississippi at Donaldsonville, 77 miles above New Orleans, and flowing thence through the Atchafalaya basin to the Gulf. It upper portion, for a distance of some 65 miles, has been leveed for sixty years. In its lower portion, overflow is unrestrained. Its upper portion has maintained its depth unimpaired. In its lower portion the deposit of sediment has so raised its bed and obstructed its discharge that it has become necessary to build the levees, at and near their lower termini, higher and higher in order to resist the floods, until now they extend 8 feet above their original height; while those above, where the stream has been kept within its channel, remain as they were fifty years ago.

R. S. Taylor: Testimony before River and Harbor Committee, February, 19, 1902—

The Mississippi River is to-day one of the best alluvial rivers for navigation, because it is one of the largest, and it is the average size that makes it. If you confine a river within a definite channel at all stages, you make it in effect that much larger river, and you give it that much more power, and it makes that much better channel. . . .

The inclosure of the flood water in the channel makes the river larger. It sets at work a greater amount of energy in the scouring-out process, and makes the river throughout its length a river of

larger average size, and capable of doing more work, and capable of keeping open a better channel.

We have made many studies in years past of this question. In the early part of the work we made numerous test surveys at points where there had been old crevasses of long standing. We first made a survey for some miles above and below the crevasse and then closed the crevasse, and then made another survey some time afterward over the same stretch of river. We always found that the river had been improved by the closure of the crevasse; improved in some cases as much as 15 per cent.

Gen. C. B. Comstock, before House Committee on Levees and Improvement of the Mississippi River, in 1882—

Question: Would that plan (outlet at Lake Borgne) have a good or bad effect, to draw off the surplus water in that way when it gets above the manageable point?

Answer by General Comstock: To draw off water enough there to produce any important relief in case of floods, I doubt very greatly whether it would not be rather injurious than beneficial. I think the only permanent relief is to make big levees.

Hearings before the Senate Committee on Commerce, May, 1890—

Gen. C. B. Comstock:

It may be concluded then, that the reduction, by any large amount, of the flow of the Mississippi at Lake Borgne, below what it has been for many years, will be ultimately followed by a rise in the flood heights at that place and a shoaling of the river below and at its mouth. . . .

(High water opening.) That is about the level of the banks. For that purpose it would give some relief. But at Lake Borgne the rise and fall of the river is only some 12 feet, perhaps, so that if you stop your sills at 9 or 10 feet above low water it would require one a good many miles long, which would require expensive work.

Question: How deep an outlet could you have there without shoaling up the river below?

Answer: It is possible you might take out 100,000 second feet. You might take 100,000 second feet without injuring South Pass. I do not think you would gain enough by that experiment to balance the danger you run.

(Damage likely to New Orleans and vicinity by Lake Borgne Outlet.) "It would raise the water there by a number of feet" and necessitate the leveeing in of the whole rear of the city.

Lieut. Col. C. R. Suter:

The so-called outlet plan was condemned (by the Commission)

in toto. In some places there is very little question that the navigation of the river has been seriously deteriorated by the existence of breaks in the levees. . . .

(Opinion as to Lake Borgne Outlet.) I have not changed my mind about it at all since the first report of the Commission was made. I think it is a perfect piece of foolishness. Because, in the first place, it would do not good, and in the second place, I feel very confident that it would do a great deal of harm. In the third place, I do not think it could be maintained even if it was opened.

Question: If you relieve the river of a large amount of water, it would do good at very high water, would it not?

No, sir; I do not think so. I have very grave doubts about it. When it was first opened it might, but in a very few years this relief would disappear entirely. I think the floods above the outlet would ultimately increase. Wherever you divide the channel you must have an increased head to carry the water through the two branches. That is the smaller river has the higher slope. . . . (Instead of using a waste weir), I would rise the levee beyond what the water would have been if the waste had not been there.

It has been known, ever since hydraulic laws have been formulated, that if you have a stream of given capacity and divide it, you must have a steeper slope to carry the water off than you had when it was one stream. That can only be done by raising the water surface above the point of division. The velocity is checked by the increased resistance in the two branches, and you must have an increased slope to force the water through.

The general idea is that where crevasses occur the velocity is checked and the movement of the flood wave is retarded, and this retardation of velocity has a tendency to increase the flood height. The water behind keeps piling up on that in front until you get 4 or 5 feet of abnormal elevation.

I think that the water that is drawn off into the St. Francis Basin and then returned at Helena will give a greater height at Helena than if that water were to pass down the main channel. The same phenomena occurs at Vicksburg, and possibly at other places.

Maj. B. M. Harrod—

A great many crevasses were open when we first took charge of the river. We surveyed those while they were open in such cases as Bonnet Carre and Morganza, knowing they were to be closed. After closing, we resurveyed and we found in both of those instances a large increase in section immediately below. For instance, we surveyed the river in the neighborhood of Morganza, this crevasse being open in 1883. Up to the time of closing that crevasse, there had been a progressive loss of section below the Morganza Crevasse of some 25 per cent. That crevasse was then closed, and in the ensuing year there was recovered in section about 12 per cent.

Pretty much the same thing is true at Bonnet Carre. Then again, up in Mississippi, we reversed the process. We made our surveys on the Mississippi front when the Riverton, Bolivar, and Lake Charles levees were up. During the flood of 1882 crevasses occurred at these points. We resurveyed subsequently to the crevasses and found in Lake Charles, below those crevasses, I think a loss of 30 per cent, and a smaller loss at Riverton and Bolivar. After these breaks were closed and the levees rebuilt, we resurveyed and found a restoration of section by the restoration of the volume of water to the river. Whenever the Commission had an opportunity to make a test of that sort, they always applied it, and there is no exception to the results. A breaking of the crevasse and an outlet of that character have always resulted in the loss of section, loss of capacity, of discharge, and closing the outlet has restored it.

That Lake Borgne Outlet would discharge over a tract of land about 6 miles wide into Lake Borgne, which has a depth of 7 or 8 feet. The distance from the river through Lake Borgne to such deep water as you find off the mouths of the Passes in some 60 miles. Although Lake Borgne is now at tide level, and the tide ebbs and flows there, that condition would immediately cease when the outlet would be made, and you would then have a fall of 15 feet at high water—the height of the river above the Gulf—distributed over 60 miles from where the Lake Borgne outlet is located to deep water in the Gulf. That distribution of slope means inevitably the elevation of the surface of Lake Borgne. I would estimate that when the channel was built through it, it would amount unquestionably, in my opinion, to 5 or 6 feet. Lake Pontchartrain empties into Lake Borgne. Therefore, if you raise the level at Lake Borgne you raise the level to Lake Pontchartrain.

Now, if Lake Pontchartrain were raised 4 or 5 feet, nineteen-twentieths of the city of New Orleans would be probably 3 or 4 feet below that water surface, and would need constant protection.

The city of New Orleans is thickly built up along the alluvial slope of the river bank, and within a distance of three-fourths of a mile from the river bank you reach practically a tidal marsh. If Lake Pontchartrain be raised a foot or 2 feet or more, it places part of the city that much under water. At present Lake Pontchartrain is 1½ feet above its natural level, from the Nita Crevasse above the city of New Orleans. In its normal condition some parts of the city are 1½ feet below, and it would devolve, I suppose, on the Government to protect the city of New Orleans and generally the lake bank against overflows.

I think Lake Borgne would fill up with sediment with a number of tortuous and worthless channels through it. It would become a marsh intersected with bayous, tortuous and shallow.

There is a great difference in the effect of outlets in lowering the flood surface of the river. You get a certain amount of relief from a crevasse. That is indisputable. In the case of the Nita Crevasse

the flood was lowered 1½ feet. You got this local temporary relief. I do not think there is any reason to expect that such relief would be permanent.

From 1874 to 1882, two enormous crevasses existed between Red River and New Orleans—viz, Morganza and Bonnet Carre. The maximum flood discharge at Red River Landing during the flood of 1882 was 1,595,000 cubic feet per second, and 921,000 cubic feet reached New Orleans. So there was lost through the two crevasses 674,000 cubic feet, yet the gauge at New Orleans read 15 feet, an extremely dangerous stage, during the flood. Although nearly half of the flood volume had been lost through the crevasses, the gauge at New Orleans stood within a foot of the highest known.

The logical conclusion from this is that during the eight years that intervened between 1874 and 1882, the river had adjusted itself to the condition of these outlets or crevasses and had lost the local and temporary relief that they gave at first. . . . Its effect if it had gone on for a longer time, I have no doubt would be reversed from a relief to an injury.

Question: Do you think it practicable and possible to build levees at such height at a reasonable expense, so that extreme high water will carry the whole volume of the Mississippi River in that one channel?

Answer: Yes, sir; unquestionably.

Question: You think . . . it is absolutely certain that the river can be confined in that one channel, and that one channel would carry the entire volume of water?

Answer: I have no doubt of it. I have no doubt that to increase the height of the levees is the cheapest way of doing it. If you build outlets, you are obliged to protect the country back of those outlets.

They talk about an escape through outlets of 400,000 and 500,000 cubic feet per second. That is, you would have to make two channels each about the size of Red River in great flood, and take them through a cultivated country 100 miles to the sea, and carry them through that country without hurting anybody. The actual work, cost and risk of raising those levees to the necessary height to carry the entire volume in a single channel are less in amount than would be required to make the channels for outlets and confine their discharges to those channels.

Question: Suppose the water be carried off in some other direction than the river itself—its surplus water?

Answer: I do not believe that would be a permanent result. There might be immediate relief, but the ultimate effect would be to raise the flood surface. . . .

I do not think the outlet would afford valuable relief under any circumstances. If it afforded very little relief it would do very little injury. It would fill up the mouth in proportion to the amount abstracted, and certainly the consequential damages or what would be required to protect the city of New Orleans from the back water in Lake Pontchartrain is a very important subject.

Col. Henry Flad:

I am opposed to the opening of an outlet like that now proposed to be made at Lake Borgne. It would give temporary relief only. The outlet would probably silt up. If it would increase in depth and in quantity of discharge it would interfere with the efficiency of the jetties in scouring a deep channel at the mouth of the South Pass.

Maj. H. B. Richardson, Chief Louisiana State Engineer—

Question: What would be the effect of an outlet at Lake Borgne?

Answer: I think the effect of an outlet at Lake Borgne, if it were made into a previously prepared channel at once, would be similar to the effect of a crevasse, which would produce an immediate temporary and local relief to the flood condition of the river. Its final effect, I believe, would be to silt up Lake Borgne as any other crevasse silts up the country in its neighborhood, or the lakes or country into which it discharges, and that such a channel could not be maintained for many years. If it were maintained it would necessarily result, in consequence of the division of the channel into two parts, in the manner which has been frequently stated here yesterday and to-day, in a deterioration of the channel below, a deterioration of both channels. If the channel through Lake Borgne were maintained, the present channel of the river would be deteriorated. My belief is that the Lake Borgne channel would close before a very long time. It would result in the meantime in the elevation of the bed of the river below, and consequently its flood levels above and below.

Question: What is the effect of the Nita Crevasse?

Answer: The first effect of a local crevasse is a considerable depression of the flood level of the river. Its effects certainly become less and less with years, partly due to the silting of the crevasses themselves generally; I believe also partly to the filling up of the bed of the river below them. . . . As regards protection from overflow, I am convinced there is no other system than that of building levees along the banks of the river.

Capt. Smith S. Leach:

To start with what should not be done, I would mention the project of taking off any portion of the natural discharge of the river at any stage whatever or for any purpose, or at any point.

The salient point in connection with that topic is, first the question of the effect upon the channel of the river above and below, resulting from taking off such water under such circumstances. . . . This single trunk channel, as it approaches the Head of the Passes, is one of the finest navigable flowing streams on the face of the earth. It is of reasonable width, very deep and has at all times a regular and moderate current.

It divides into three principal branches, each of which is nar-

rower and shallower and more irregular in its regimen than is the main stream. Here the phenomenon is presented of a large, deep and good channel being transferred into three narrow, shallow and poor channels. This phenomenon occurs at the corresponding point on every known alluvial sedimentary stream on the face of the earth that branches into a delta formation.

Now, if the degradation of these subsidiary channels occurs here where the stream divides into three parts, why will it not occur at Lake Borgne if you there divide it into two? The fact can be explained on no other hypothesis than that in division there is weakness, a proverb more familiar in the inverse terms, "in union there is strength."

I never heard any other advanced for it, and its converse, which the outlet theory demands for its support, is not only absurd on its face, but contradicts every fact of the river's life which has come to my knowledge. Anything further on the subject of outlets is only an elaboration of that general statement. To substantiate that, if proof should be necessary, there are frequent observations in the bed of the river itself. These consist of a large number of exact measurements, as precise and accurate for that purpose as would be any measurement that could be made of the length of this capitol building. We have not guessed at this thing; we have measured it, and we had no theory to establish when we made the measurement, but we made the measurement for the purpose of finding out the theory.

The measurements have shown conclusively, in repeated instances, that when a crevasse occurs the channel for a few miles immediately below becomes distinctly smaller. When that crevasse is closed, measurements made before and after the closure have shown that this loss in the area of the channel is recovered. When a crevasse opens and immediately after that opening, by exact measurement there is found to be a deterioration of the channel of about 12 per cent of its area, and again when this same crevasse is closed after the next succeeding flood, there is found to be a recovery of this 12 per cent lost. I do not think any other hypothesis will explain it.

Such floods as Colonel Ellet anticipated (stage of about 64 feet at Red River Landing) have never been realized; never will be; never can be.

The whole tenor of his report shows very plainly that he was forced to accept the outlet theory against his deeper conviction, simply because he thought no levees which could possibly be built would restrain the floods which he expected in the future, due to the progress of deforesting and the extension of cultivation and drainage, together with the building of levees.

His idea was that the extension of the levees would hurry forward the discharge from above. The elevation of the flood line would begin at Cairo and increase rapidly as it proceeded downstream. Subsequent experience has shown that these apprehensions were entirely unfounded.

The super-elevation of the flood surface within the history of the Po for several hundred years, due to the extension of levees and other causes, is about 3 feet.

The best method of controlling floods is by means of levees. There are physical limits to the building of levees, and if a flood were so high as to exceed these limits, then it would be necessary to obtain relief. It was under such apprehension, in my opinion, that Colonel Ellet proposed an outlet.

Senator Washburn:

Major Suter takes the position that the river with these outlets would not discharge the water as rapidly as though it were held in one channel.

Captain Leach:

I think it is fully agreed that there would be immediate relief. . . . It would afford slight local relief from the pending flood, undoubtedly. . . . It would reduce the level slightly. . . . The divided channels must inevitably take a higher slope and in doing so the point of their divergence must be elevated absolutely. And it will raise the flood line. Nothing else you can do will elevate the flood plane so certainly. In fact, that is the one solitary thing that must give New Orleans bigger floods than ever before.

Question: With an inflow of 2,100,000 second-feet and an outflow of 1,100,000 at the mouth of the river, how would you prevent overflows?

Answer: By one of the best known principles of river physics; that is, that there is a very appreciable reservoir effect in the volume of the channel itself.

If the water is flowing in at Cairo faster than it is flowing out at New Orleans, and I am asked where the surplus goes, I am able to reply that it goes to raising the surface of that water. There are thousands of square miles of water to be raised and it rises sometimes in places as high as 2 or 3 feet a day.

The main question presented is one which has occupied the serious attention of the ablest engineers of several nationalities for a century.

Divested of all theories it is, What is the best method of conducting a sedimentary river from the continent into the sea? Taking into view long periods of time, it may be broadly stated that the rivers are washing the continents into the oceans. There is no method by which this process can be stopped. It is gradual but nevertheless certain, and the oceans must for all time continue to be reservoirs for the reception of sediment discharged from the turbid rivers.

The security of a population living in an alluvial basin subject to overflow, demands first of all stability in the regimen of the stream, particularly as regards the elevation of the flood plane.

Nature, working in her gradual, silent, but always efficacious way, has been engaged for centuries in an attempt to realize the very conditions which we have seen are essential to the permanent discharge of the river into the Gulf with safety to the inhabitants, and ample facilities for navigation. . . . Nature's effort has always been in the right direction, and to-day she has reached a point little short of perfection.

To divert from this channel at a point 80 miles above its mouth, any part of the annual flow of the river, is to abandon that much of this magnificient conduit. . . . To substitute for a permanent flood plane in the vicinity of New Orleans one which will rapidly advance in height. . . . That the flood plane will be suddently lowered is true, and it is no less certainly true that it will immediately begin to advance. . . .

No fact of hydraulic science is more firmly established than that the loss of volume of a sedimentary stream through lateral outlets is accompanied by deposits in the main channel below the outlet. It is an universal law running through all branches of hydraulic engineering. . . . In an irrigating canal the section of the main canal is reduced in passing the mouth of each side canal by an amount proportioned to the volume there abstracted. Were the main canal continued at full size, and the water charged with sediment, it would soon be obstructed with deposits. The same principle is recognized in questions of pipe distribution for water and sewerage. The size of the conduit must be everywhere proportioned to the volume of flow. To this rule, so universal, the Mississippi River is no exception. (He then cites again in detail the results of measurements above and below crevasses, and then continues.) With these facts in view, the conclusion can not be avoided, that lateral outlets from the Mississippi River do cause a deterioration of the channel below and are therefore injurious.

It may be confidently stated that the effect of an outlet will only extend from a few miles above its site to the mouth of the river; that within these limits they will be temporary and injurious; that nature's effort to keep the river in a single channel from its source to the deep sea is the only road to perfect and permanent conditions and should be aided and not thwarted by man.

I have arrived at the conclusion that if an outlet is made in the vicinity of New Orleans, the velocity at that place would be increased about 25 per cent. At the present time, with the present velocity, it requires works of the greatest magnitude and the most formidable character known to this branch of engineering to prevent the banks in front of New Orleans from caving into the river. If the velocity were increased 25 per cent, no works that are known and no amount of money would keep these banks from caving. . . . The outlet would be in action not more than one hundred days per year. During the other 265 days the river, flowing through a channel which is larger than it demands, would fill up.

Capt. Dan C. Kingman, then in charge of the Fourth District:

The present discussion in regard to the Mississippi River has been relative to the effect of outlets on flood heights, and I have here a hydrograph which shows what outlets actually do to the river by showing what the crevasses which have taken place during this last flood have done to the flood heights.

The river was at a high stage on the 1st of March at New Orleans, and continued to discharge at a very rapid rate, amounting to 1,280,000 second-feet on March 11 and 12. Up to that time, no breaks had occurred when the levee broke at Nita Plantation. After it had been running eight or ten days, the measured discharge was 90,000 second-feet, the crevasse being about 600 feet wide and 15 feet deep. In spite of all efforts with piling and mattresses, it increased to width of 3,000 feet and a discharge of 400,000 cubic feet per second. So large an outlet as this ought to have produced a very marked effect on the river. If any outlet would do any good, surely this one, discharging 400,000 cubic feet per second, ought to produce great relief.

At College Point, a few miles below the crevasse, the stage of the river fell about 1½ feet and about 1 foot at Carrollton. At Plaquemine, 50 miles above, there was no preceptible effect due to the crevasse.

Crevasses on the right bank between Bayou Sara and the mouth of Red River, aggregating 237,000 cubic feet per second, did produce some effect, the fall at Bayou Sara amounting to about a foot and at Red River the fall the first day amounted to 2 or 3 inches, and in three or four days the fall amounted to a foot.

Question: What is your conclusion about the outlet system at Lake Borgne?

Answer: It would be a disappointment. It would not afford the relief which is counted on; it would be an entire failure as a means of relief from overflow.

If it did any good at all, temporarily, it might do good from the outlet down to the mouth of the river. . . . Finally, I do not see why there should be any more relief from the Lake Borgne outlet than there was at Plaquemine from the Nita crevasse. I think we have an absolute demonstration of what an outlet will do. Here is a crevasse that is now flowing and this is what it has done. Below the crevasse it has afforded a little relief but nothing like commensurate with the amount of water taken out, and above the crevasse it has afforded no relief at all. . . .

The Mississippi River flows in a channel of its own formation. . . . This channel has a certain size. It is of a size sufficient to carry its average flood discharges. Whenever it has to carry a discharge greater than the average flood discharge it is overtaxed. If it were to carry only half the water it does now it would have only its present capacity.

The Morganza Crevasse occurred in 1874 and remained open as a crevasse practically until closed in the winter of 1886-1887.

a period of about twelve years. It has a deep bend there and plenty of water, and there had been no trouble with the navigation until after the crevasse was formed. Then navigation grew worse and worse; the channel became so narrow that it was difficult for two steamboats to pass. Since the closure of the crevasse conditions steadily improved and the normal conditions before the break, were restored.

Similar changes in channel capacity were noted at Bonnet Carre.

Report M. R. C., 1890, page 3305, Captain Kingman further says:

It therefore may be seen that the influence of these enormous outlets (crevasses) thus suddenly let loose at the flood height was something, to be sure, but it was very insignificant in comparison with the amount of water drawn from the river. It should also be noted that its effect is almost entirely felt below the points at which the crevasses occurred, and that at a comparatively few miles above the crevasses the effect ceases and amounts to nothing. There could not be a more forcible object lesson than this upon the worthlessness of outlets as a means of controlling floods.

M. R. C. Report 1890, page 3304:

Nita Crevasse, 2,892 feet wide; mean velocity, 9.28 ft. per sec.; mean depth, 15 feet; discharge, 402,556 cubic feet per sec.; lowered gauge at College Point, 5 miles below, 1.9 feet, and at Carrollton, 1 foot.

SACRAMENTO RIVER.

Reclamation of Lands Subject to Overflow.

In 1880, a Board of Engineers consisting of Gen. B. S. Alexander, Col. George H. Mendell, and James B. Eads, recommended that the entire flow of the river be confined between high levees; temporary spillways or relief canals to carry the excess volume, but these to be abandoned when the river had developed its ability under this treatment to carry all floods.

In 1895, another Board of Consulting Engineers (Messrs. Marsden and C. E. Grunskey) recommended a plan whereby the surplus flood water, beyond a stated amount, should flow over long weirs into side relief channels, the river proper carrying only about one-third of the flood volume.

In 1904, a board consisting of T. G. Dabney, H. B. Richardson, M. A. Nurse, and Maj. H. M. Chittenden, proposed a plan whereby the entire flow at high stages should be confined by permanent levees, utilizing temporary side relief channels until project was sufficiently advanced to carry entire flow.

This board, commenting on the spillway or by-pass plan of 1895, says:

The Commission finds itself unable to accept this plan as a permanent solution of the Sacramento River problem. . . . The concentration of outflow at a few points as contemplated in the by-pass system, must, in the opinion of the Commission, fail as an adequate remedy for existing evils and result in continuance of unsatisfactory conditions.

It is a matter of common observation that when, in a silt-bearing stream, a large volume of flood water is permitted to escape at one point, the resultant diminution of current energy in the channel at the point of outflow causes a marked shoaling of the river immediately below by deposit of sediment that the more enfeebled current is unable to transport.

There is also a strong probability that the passage of large volumes of water through wide and shallow channels may result in silting up these artificial waterways in their upper reaches and thus impair their utility.

In 1905, all of the reports were reviewed by a Board of Engineer Officers, consisting of Col. William H. Heuer, Col. Thomas H. Handbury, and Capt. William W. Harts. Their report states:

By the 1895 plan, much of the sediment at flood stages would inevitably be carried into the side relief channels, where, with the reduced velocities, much of it would be deposited out of the river bed; but on the other hand, for the same reason, the scour necessary to maintain good navigable channels in the river bed can not be relied on. . . .

The adopted plan for the control of floods in the river systems of the Sacramento Valley, as reported in 1910 by a board consisting of Lieut. Col. John Biddle, Capt. Thos. H. Jackson, and Lieut. Charles T. Leeds, provides a system of levees of such height and so located as to care for small floods, the greater part of maximum floods to be diverted over four weirs or spillways into auxiliary channels, the flow from which returns again to the main river, all involving an estimated expenditure of $33,800,000, of which $10,440,000 is for levees and the balance for weirs and side channels. The amount previously expended for reclamation, flood control, and navigation equals $19,157,441. This applies to about 250 miles of river.

(The estimated cost of one weir or spillway 8,000 feet long is $1,000,000.)

Their report says:

The by-passes will not be utilized at all some seasons, and it is

probable that they will be in commission but a short period other years; therefore, the deposit, if any, should not be great . . .

Further, there will be no water escaping into the by-passes except at stages above about 6 to 8 feet below the present maximum flood stages. Therefore, the material carried into the by-passes should be principally material in suspension, which will not be deposited in them. It may be argued that the use of the by-passes will result in deposit in the river immediately below the spillways in the portion of the river carrying large quantities of mining débris. . . . The waters of the river after the flood wave has passed will be kept in the present channel, where they will be utilized to scour out any deposits that may have formed during the flood.

(It will be observed that this conclusion does not agree with the hydraulic law quoted in the preceding pages.)

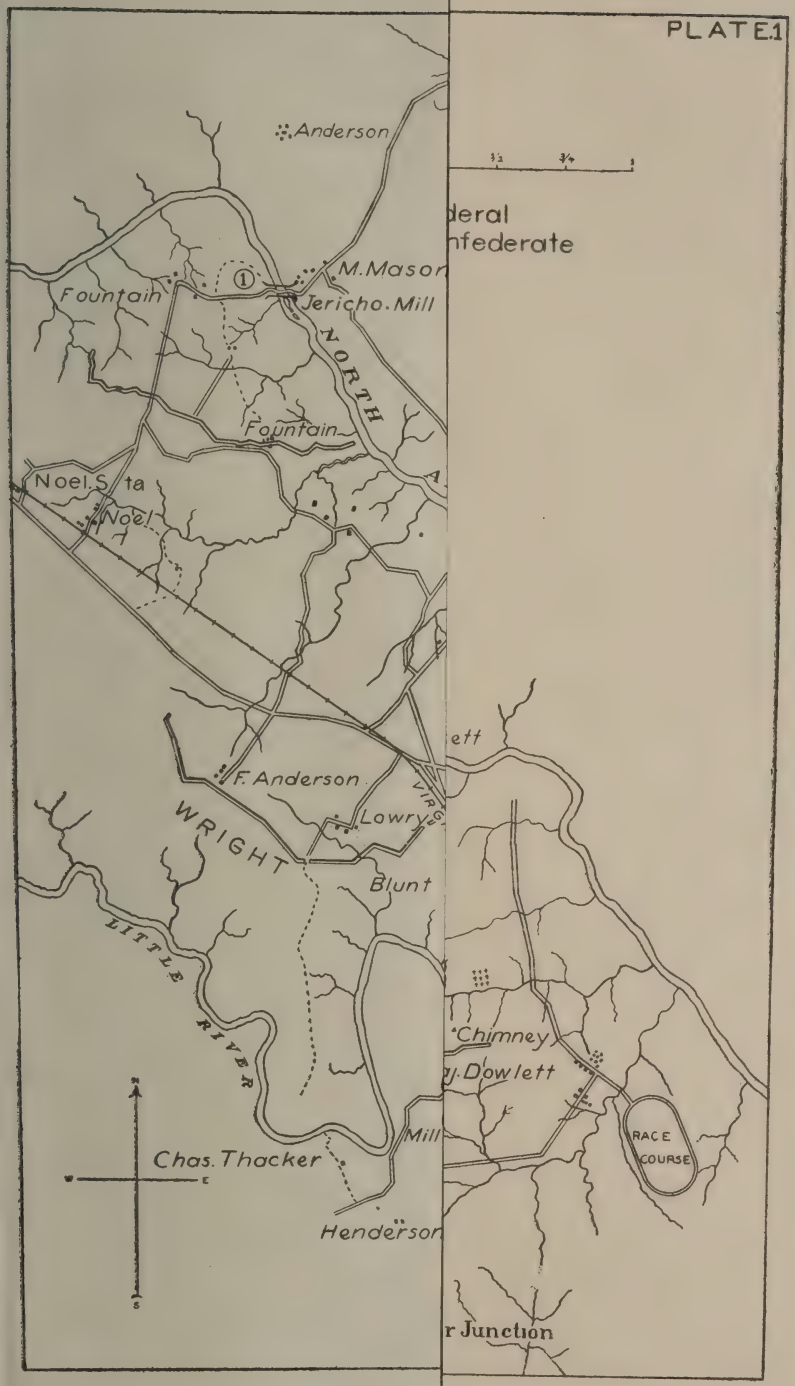


PLATE 1
Anderson
deral
nfederate
M. Mason
Fountain
Jericho. Mill
NORTH
Fountain
Noel. S ta
Noel
F. Anderson
Lowry
WRIGHT
Blunt
LITTLE RIVER
Chimney
y. Dowlett
RACE COURSE
Mill
Chas. Thacker
Henderson
r Junction

PLATE.1

Mt. Carmel
C. H.
Anderson
Quallers
Hackett
Kelly
Federal
Confederate
M. Mason
Fountain
Jericho. Mill
Hangrave
Worthen
NORTH ANNA
Fountain
A.R. Riggs
Noel Sta
Noel
Quarle
Quarles. Mill
Madison
RIVER
Matthews
J.A. Matthews
BURNSIDE
Chandler
Hackett
WARREN
Fall. Mill
Bibb.
Oxford
F Anderson
Lowry
VIRGINIA CENTRAL R.R.
WRIGHT
Blunt
W. Cox
HANCOCK
Lowry
Ch. Cady
Sta.
Chimney
Maj Dowlett
Montgomery
LITTLE RIVER
A Dowswell
New Market
Mill
Chas Thacker
Burroughs
D. Miller
RACE COURSE
Home
Henderson
Mallory
Hanover Junction

An Historical Example of Forcing a River Crossing: The Federal Cavalry at the Pamunkey River, Va., May 27, 1864.

BY

Capt. R. P. HOWELL, Jr.,
Corps of Engineers, U. S. Army.

In the course of the movement from the Rapidan River to the James River, May-June, 1864, the Federal "Army of the Potomac" found itself, on May 25, confronting the Confederate "Army of Northern Virginia" in approximately the situation shown in Plate I. It will be seen that the North Anna River split the Union Army into three parts, thus increasing the necessity for numerous stream crossings and good lateral communications. All these bridges, except the permanent ones, were constructed by the Fiftieth New York Volunteer Engineers, under Lieut. Col. Ira Spaulding, and were located as follows, in order downstream:

1. *At Jericho Mill.* A canvas ponton bridge, 160 feet long (in addition to a ford, not practicable for artillery).
2. *At Quarles Mill.* A wooden ponton bridge, 80 feet long, and 220 feet of crib-bridge.
3. *At ford below Quarles Mill.* A canvas ponton bridge, 100 feet long.
4. *Below Ox Ford.* Corduroy bridge.
5. Telegraph Bridge (permanent highway bridge).
6. Railroad Bridge.
7. *At Railroad Bridge.* Two canvas ponton bridges, each 100 feet long.

On the evening of the 25th, a train of eight wooden boats, under Captain McDonald, was at Wrights Tavern, some 8 or 10 miles north of the Federal lines. During the night of the 25th-26th, the ponton portion of the bridge at Quarles Mill was replaced by a permanent structure, thus releasing another train of eight wooden boats, under Major Beers. The remainder of the ponton equipage of the Army of the Potomac, consisting of a train of eight wooden boats under Captain Personius, was at Mount Carmel Church.

Finding General Lee's position stronger than he cared to attack, General Grant decided to continue the movement around the Confederate right, crossing the Pamunkey River near Hanovertown. (See Plate II.) The crossing was to be seized by two divisions of General Sheridan's cavalry corps, now at Pole Cat Station. The cavalry was to be accompanied by the two canvas ponton trains of twelve boats each, for throwing the bridges. The First Division of the Sixth Corps was to start early on the morning of the 26th, escort the trains to Chesterfield Station, and then follow the cavalry to and across the Pamunkey.

Now, however, a difficulty arose. In order to deceive the Confederates in regard to the contemplated move, the infantry divisions were not to commence the withdrawal to the north bank of the North Anna until the night of the 26th, whereas the advance movement by the cavalry was to start from Chesterfield Station on the morning of the 26th, and it was manifestly inadvisable to remove the bridges while the infantry were still on the south side of the river. Accordingly, very early on the morning of the 26th, Colonel Spaulding ordered Major Beers to move his wooden train from Quarles Mill to Jericho Mill and release the canvas train under Captain Van Brocklin; and Captain McDonald to move from Wrights tavern to the railroad bridge and release the remainder of the canvas equipage under Captain Folwell.

By 7.00 a. m., Major Beers had built a wooden bridge at Jericho Mill, and Captain Van Brocklin, with his canvas boats, was on his way to join the cavalry. General Hancock very naturally declined to permit the bridges in his rear to be removed until others arrived to take their places; but Captain McDonald reached the railroad about 11.00 a. m., and by noon Captain Folwell was also on the way with his canvas boats to join the cavalry. These trains marched during the afternoon of the 26th, and all night of the 26th-27th, with one rest, reaching the Pamunkey near Hanovertown about daylight on the 27th.

On approaching the stream, it was discovered that the opposite bank was held by Confederate pickets, later determined to be dismounted cavalry. Experience had proven that it was out of the question to construct a bridge under these conditions. Preparations were at once made to dislodge the enemy. Two boats were assembled out of range of the hostile fire, carried on the backs of the pontoniers to the river, and launched under cover near the point of contemplated crossing. One regiment of General Custer's

LIBRARY OF THE UNIVERSITY

PLATE II
N
Scale of Miles
Chulesburgh
Pole Cat Station
Pole Cat
North Anna
Island
Jericho Br
Chesterfield Station
Ford
Chesterfield
Junction
Taylorsville Station
Hanover Station
Hanover Court
Ashland Sta
Ground Squirrel Br
Winstons Br
Atlees Station
Coal Pits
Wilkinsons Br
Meadow Br
Mechanicsville
Walnut Grove
Cold Harbor
Shelly Grove
Stony River
Brook
Run
Herring Creek
Dorrell Cr
Taylors Fd
Governors Swp
Reeds Swp
Boot Swp
Malden Swp
Bull Swp
Munday Sp
Mattaponi River
Chapel Bridge
Forge Mill
Clarksville
Dunkirk
Small C
Watkerton
Brandywine
Hanovertown
King William C.H
Newcastle
Ferry
Piping Tree Fd
Totopotomoy Cr
Mountain Cr
Chickahominy River
Hopewell Church
Turkeytail St
Water House
Baltimore Cr Rd
Dispatch St
Broad
Rocketts
Manchester
Robinet Sta
Coal Mines
Richmond
James River and Kanawha Canal
Virginia Central R R
Richmond and York R R
Richmond and Potomac R R
Richmond and Potomac R R
Virginia Central R R
Mattaponi River
Mattaponi R R

brigade, dismounted on the near bank, then drove the Confederate pickets from the immediate vicinity of the far bank. Under cover of this fire, Captain Van Brocklin's men ferried over dismounted cavalrymen in the two canvas boats, until a firing line of some two squadrons was built up on the opposite bank. Under this protection, construction proceeded promptly, and without casualty, though occasional shots were fired at the pontoniers. The bridge, which was about 180 feet long, was completed in about an hour, and General Torbet's cavalry division commenced crossing at once.

By 7.00 a. m. Captain Folwell had gotten his train up to the river and had commenced to lay an additional bridge, a few yards above the first. This second bridge, which was 164 feet long, was likewise constructed in about an hour. These two bridges gave ample means of crossing, so that by the time the head of the infantry division began to arrive, both cavalry divisions were across.

After crossing the Pamunkey the cavalry moved forward, and, meeting the Confederates near Haws Shop, defeated it after a severe engagement.

On the evening of the 26th, the main army commenced to withdraw to the north bank of the North Anna, and to follow the cavalry to the Pamunkey crossing. About 1.00 a. m. on the 27th, all the troops which had to use the bridge at Jericho Mill were across, and Major Beers dismantled his wooden bridge there, and started for the Pamunkey, halting about midnight. As soon as the Second Corps had completed its crossing, Captain McDonald dismantled his bridge, and took up the march, joining Major Beers on the way. The permanent bridges and the railway bridge were destroyed.

The victory of the cavalry insured the safety of the Pamunkey in that vicinity, so that the army could cross at the most convenient point. While the two canvas bridges were sufficient for the advanced troops, it was evident that they would be inadequate to pass over the infantry and artillery promptly; so Colonel Spaulding, with characteristic energy, began to cast about for additional means of crossing. As the two wooden trains had remained in position until all the army was across the North Anna, it was impossible that they should arrive ahead of all the foot troops and provide new crossings for them over the Pamunkey, without delaying the march. And the third train of wooden boats, under Captain Personius, had been ordered to Butlers Mill, on the North Anna, to hold open a crossing for the Third Cavalry Division,

under Gen. James H. Wilson, which was making a demonstration against the Confederate left.

However, there was some canvas equipage remaining after the completion of the first two bridges, sufficient for a somewhat shorter span than those. Reconnaissance developed a suitable site near Mrs. Nelson's, and at 6.00 a. m. on the 28th a canvas crossing was thrown here, the bridge of 146 feet being completed in an hour. Colonel Spaulding says of this structure:

> Owing to the scarcity of material this bridge was built in long spans of 21 feet, the balks alternating on the gunwales, and over this seemingly frail bridge passed without accident two divisions of the Sixth Corps with their artillery.

The wooden equipage reached the Pamunkey at Mrs. Nelson's at 3.00 p. m. on the 28th, and Major Beers immediately threw a wooden bridge across the river below the canvas bridge. Over these four bridges the main Army of the Potomac passed safely and promptly to the south side of the Pamunkey.

At 9.00 p. m. I directed Major Ford to send Captain McDonald across the river with his bridge train, thence to Hanovertown, and throw a wooden bridge across the river in place of Captain Van Brocklin's bridge, which was to be dismantled. This was done during the night, and Captain Van Brocklin's train was parked on the south bank of the river ready to move at short notice. —Colonel Spaulding.

COMMENT.

It may be stated as a tactical principle that it is impracticable to construct a bridge so long as the enemy is in undisturbed possession of the opposite bank. There are numerous historical examples where such efforts have met failure. A familiar one is the attempt to construct the bridge at Fredericksburg on December 11, 1862. Here even artillery fire was unable to drive away the enemy's sharpshooters, hidden on the opposite bank, and it was necessary to ferry over landing parties before the construction could be continued. With the defender's artillery in concealed positions, the construction of a bridge in the face of the enemy is now-a-days even a more serious undertaking than formerly—as would appear from the fragmentary reports received from the present European war.

It will be necessary to resort to tactics very similar to the one used on the occasion in question. Infantry, and, if practicable, artillery fire will be directed on the locality of the intended crossing; ponton boats will be launched at a distance from that

locality, preferably upstream if the current is swift, and dismounted men ferried across to effect a lodgment on the opposite bank. Having secured possession of the far side and driven the enemy back so that he is unable to bring accurate fire to bear on the intended crossing, the construction of the bridge may be commenced with impunity.

The special suitability of the canvas, or advance guard, ponton equipage for rapid movements, was shown in this operation. These two bridges were doing service in rear of the Second and Fifth Corps, respectively, early on the morning of the 26th. Yet they were dismantled and loaded, and marched to Chesterfield Station and to the Pamunkey at Hanovertown by daylight on the 27th. In this movement, the trains marched, according to Colonel Spaulding, "during the afternoon and all the night, with but one short halt," thus catching up to and keeping pace with the cavalry divisions. It is extremely doubtful if the heavier reserve equipage could have made such good time.

For ferrying across large streams, or lightering from transports, the wooden ponton is, of course, to be preferred, but this instance shows that the canvas boat also may be used at times, and that a cavalry force, accompanied by the advance guard train, need not stop simply because the opposite bank is held by hostile skirmishers. The writer has seen such a crossing successfully forced in still water, as drill and maneuver problems, and sees no reason why it could not be done in any water over which it would be practicable to build a canvas bridge.

List of Authorities Consulted.

NOTE: All references are to reports covering the period in question, which will be found in Rebellion Records, Vol. XXXVI, part 1, Serial Number 67.

Lieut. Gen. U. S. Grant, Commanding Armies of the United States, page 12.

Maj. Nath'l Michler, Corps of Engineers, Acting Chief Engineer, page 291.

Lieut. Col. Ira Spaulding, Fiftieth New York, page 304.

Maj. Gen. W. S. Hancock, Commanding Second Corps, page 318.

Maj. Gen. G. K. Warren, Commanding Fifth Corps, page 539.

Itinerary of the Sixth Corps, page 658.

Asst. Adjt. Gen. First Division Sixth Corps, page 659.

Maj. Gen. P. H. Sheridan, Commanding Cavalry Corps, page 773.

Brig. Gen. A. T. A. Torbet, Commanding First Division Cavalry Corps, page 803.

Brig. Gen. J. H. Wilson, Commanding Third Division Cavalry Corps, page 871.

Maj. Gen. A. E. Burnside, Commanding Ninth Corps, page 905.

Maps are from the atlas accompanying the Rebellion Records.

René Edward De Russy.

BY

Lieut. FREDERICK W. HERMAN,
Corps of Engineers.

René Edward De Russy was born in the French island of Haiti on February 22, 1789. Two years later his father moved with his family to New York, and soon afterward to Old Point Comfort, Va., where the family lived for many years. In these two places, New York and Old Point Comfort, General De Russy was destined to serve his country during the greater part of his life. His father, Thomas De Russy, was born in St. Malo, France, and served with distinction in the Navy of the United States during the Revolutionary War. Dr. Franklin brevetted him Lieutenant in September, 1779, for gallant conduct while serving on John Paul Jones' ship *Pallas* in the engagement with the *Countess of Scarborough*. In the War of 1812 he was again in the Government service as assistant engineer in New York, and Sacketts Harbor.

René De Russy entered the United States Military Academy on March 20, 1807, and was graduated on June 10, 1812. His brother, Lewis G. De Russy, was in the Academy for a part of this time, being graduated in 1814. This was the most discouraging period of West Point's history. In Cadet De Russy's second year the Secretary of War decided that the Military Academy was of no value to the Government, and sent the greater part of the cadets to their regiments, where they served as junior subalterns. By 1812 the eighteen or twenty cadets had all been commissioned without examination, after having received very little instruction at the Academy.

Lieutenant De Russy on being commissioned, was placed on duty as assistant engineer on the defenses of New York Harbor; later he was placed on the same duty in Sacketts Harbor, N. Y. He took part in the St. Lawrence Campaign of 1813, including the battles of French Creek and Chrystler's Field. The next year

he took part in the Lake Champlain Campaign and the battle of La Cole Mill, and was brevetted Captain on September 11, 1814, for gallant conduct in the battle of Plattsburg. After this battle he served as Chief Engineer on the staff of General Macomb, in the Lake Champlain district. He was promoted to Captain February 9, 1815.

For two years after the war Captain De Russy was on duty as assistant engineer in the construction of Fort Montgomery at Rouses Point on the Canadian border; then he was sent back to New York, first as assistant, and later as superintending engineer of the defenses of the harbor. From 1821 to 1825 he constructed coast fortifications along the Gulf of Mexico, and was again sent back to New York to his old duties until 1833. During this period he was brevetted Major for ten years faithful service in one grade (September 11, 1824) and promoted to Major of Engineers (December 22, 1830). In 1827, he had also served on the Board of Visitors of the United States Military Academy. On July 1, 1833, he assumed the duties of Superintendent of the United States Military Academy, relieving Colonel Thayer, the "Father of the Military Academy." He served as Superintendent for five years, faithfully carrying out and extending the policies of Colonel Thayer, and turned over his trust in 1838 to Major Delafield. On June 30, 1834, he was brevetted Lieutenant Colonel for faithful service and meritorious conduct.

In 1838, Colonel De Russy took charge of the construction of the fortifications of Forts Monroe and Calhoun, Virginia, and was engaged in this duty for the next sixteen years. The defenses he built at Fort Monroe held this post in the Union in 1861. In this district—which was his childhood home—he made his reputation as one of the best coast defense engineers in our history. During this period he was also in charge of harbor improvements in Delaware Bay (1838-1839), defenses of Chesapeake Bay (1838-1842), improvement of Virginia rivers (1853-1854), and a member of the Atlantic Coast Defense Board (1848-1854). In 1854 he was transferred to the Pacific Coast, as a member of the Coast Defense Board, and in charge of the fortifications of San Francisco Bay. Three years later he was ordered to Sandy Hook, and the same year sent back to Fort Monroe and redetailed on the Atlantic Coast Board of Engineers.

In 1861, Colonel De Russy was 71 years old; there being no retired list at that time he was sent back to the defenses of San

Francisco Bay, in the construction of which he spent the short remainder of his life.

Colonel De Russy was in the command of the Corps of Engineers during the three years just preceding the war. In 1857 he built the fort on Ship Island, Mississippi, and in 1860 was on the board that examined the Potomac aqueduct. From 1861 to 1865 he was superintending engineer of the San Francisco Harbor defenses and, after 1863, in charge of all Pacific Coast defenses. He was President of the Board for Land Defense of San Francisco Harbor in 1864. His last brevet—Brigadier General for long and faithful service—was given him March 13, 1865. He died while on active duty at San Francisco on November 23, 1865, aged 75 years, the oldest graduate of the Military Academy on active duty.

In his long and useful period of service, Colonel De Russy became the best fortification engineer in the Army; and in connection with this service made some improvements in artillery material, including the invention of a barbette depressing gun carriage. The War Department order, announcing his death, stated that "... for over half a century he zealously and faithfully devoted himself to the highest and most important duties of an engineer officer."

Miscellaneous Notes.

Depositing Material by Derrick Beyond Reach of Boom.

BY

Mr. M. A. Milliff.*

A great deal of trouble has been experienced on work at the Sabine-Neches guard lock, due to caving banks. The bulk of the excavation for head-bay and tail-bay lock-pits was made with a hydraulic dredge previous to the placing of the cofferdam and was carried down to —30.0, M. L. G. L.

It was the intention to use wheelbarrows in completing the excavation, but after the lock-pit had been unwatered for a short time the banks began caving and sliding into it, and it was found that excavation could not be done economically with wheelbarrows. A derrick was then used, depositing the material as far as the boom would reach.

On account of the increased excavation made necessary by slides, the dumping area provided proved insufficient and the weight of the deposited material caused further sliding and crawling of the banks.

It became imperative, therefore, that some method be devised for depositing the excavated material at a point far enough distant from the lock-pit not to cause the banks to slide. As the amount of material to be excavated was not sufficient to justify the purchase and erection of a cableway or additional derricks for this purpose, it was necessary to accomplish this with the plant available.

Very satisfactory results were obtained by rigging up a derrick with a trolley line.

A brief description of the arrangement, rigging and operation are given on the following pages.

*Superintendent, Sabine-Neches Guard Lock, Texas.

RIGGING.

Fig. 1 shows the manner in which a derrick with three-drum engine was rigged for trolley work. The symbols used for the different lines are labeled on Fig. 4.

The "load line" is shown carried on back drum. As shown on sketch, this line runs from back drum through sheave in bottom of mast, thence through sheave in end of boom and end of line made fast to top of steel block. The rigging for load line is the same as for other derrick work.

The "trolley line" is shown carried on middle drum. This line runs from middle drum through sheave in bottom of mast, thence through sheave in end of boom, thence through bottom of sheave in steel block and end of line made fast to top of gin pole.

The "boom line" is shown carried on front drum. This line runs from front drum through sheave in bottom of mast, thence through sheave in top of mast, thence through sheave in end of boom, thence back through second sheave in top of mast, etc., being rigged as a triple block with end of line dead-ended to middle of boom. The rigging for boom line is the same as for other derrick work.

Five-eighths-inch hoisting cable was used for all lines.

If a two-drum engine is to be used, the boom line can be carried on a hand crab, as only an occasional change in the position of the boom is necessary.

The block used is an ordinary single-sheave steel block with swivel hook. It is necessary to cut out the rivet or bolt in corner facing derrick, as trolley cable would rub on it during operation.

The material box is an open-end skip of 1-cubic-yard capacity, fitted with chains—one to each back corner and one to the middle of the front end. The front chain is fitted with a trip-hook. The ring in end of the chains is hooked in the hook in the block.

For the gin pole or tail tower, 26-foot yellow pine piles were used, set up on a suitable foundation to prevent sinking into the ground under strain. Old hoisting cable was used for guys.

OPERATION.

Fig. 1 shows position when picking up the loaded box.

The operator hoists the load by picking up on the load line, bringing the trolley line tight as the load is raised and swinging the derrick around so that the boom will face the gin pole.

The load is hoisted to the desired height and the trolley line

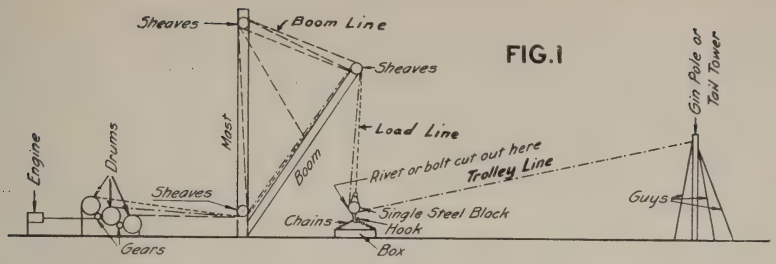


FIG. 1

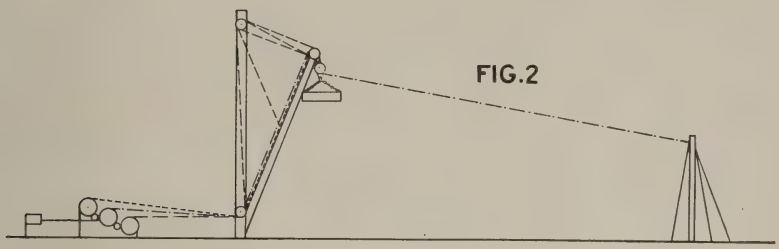


FIG. 2

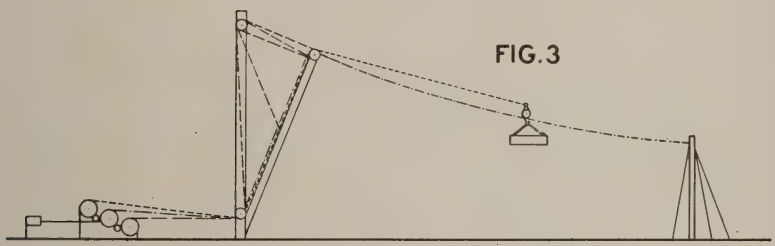


FIG. 3

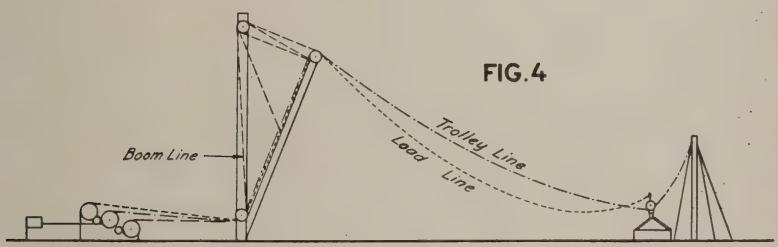


FIG. 4

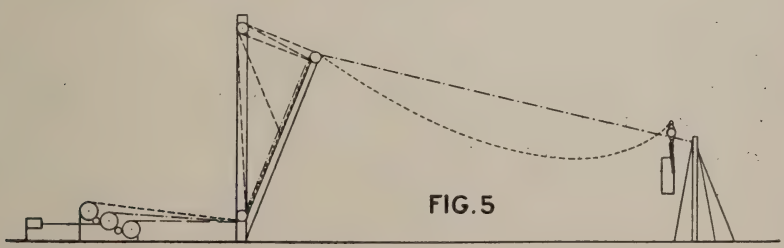


FIG. 5

tightened as shown in Fig. 2. At this point the operator holds the trolley line tight with the foot-brake and then releases friction on the drum carrying the load line, which allows the box to trolley toward the gin pole as shown in Fig. 3.

When the box is over desired dumping place the foot-brake holding the trolley line tight is released, allowing the box to drop as shown in Fig. 4.

The hook on the front chain is then tripped and the box is dumped by bringing the trolley line tight, as shown in Fig. 5.

The box is returned by pulling in on the load line and slacking off on the trolley line, bringing the box back into position for loading.

The derrick can be swung around by swinging gear or swinging engine and bull wheel as in other derrick work. It was found, however, that it was necessary to put an attachment on the end of the boom to prevent the trolley line from jumping out of the sheave when worked at an angle greater than 60 degrees each side of the line from the derrick to the gin pole.

REMARKS.

Two derricks were used on this work, both being 10-ton steel guy derricks with 90-foot masts and 75-foot booms. One derrick was equipped with a three-drum 7¼ by 10 inch hoisting engine with swinging gear attached, and the other with a two-drum 8¼ by 10 inch engine with independent swinging engine. The 8¼ by 10 inch engine did the work more satisfactorily than the smaller engine.

The trolley line arrangement has been operated for a distance of 300 feet with a 12-foot drop. The load necessary to operate it depends on the condition of the hoisting engine and the ease with which the drums overhaul, but it is believed that a 2,500-pound load will be found necessary to operate trolley on this flat slope.

In bailing slush out of a hole where five men filled the boxes with buckets and one foreman, one hoisting engineer and a laborer to dump the boxes completed the crew, 137 boxes have been moved in eight hours.

In harder digging, where it was necessary to load the boxes with shovels, an average of 70 boxes in eight hours has been maintained with the following crew:

One foreman; one hoisting engineer; and eight laborers.

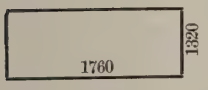
An Acreage Rule of Thumb.

Mr. G. Edward Gibson, C. E., Assistant Engineer, Barge Canal Office, Albany, N. Y., contributes the following easily remembered rule for determining very closely the acreage of areas when the principal dimensions are known. It should be of especial assistance to those who have difficulty in recalling the number of square feet in an acre. It is quicker of application than to deduce the number of square feet from some known relation and then apply it, or even to apply the number if recalled.

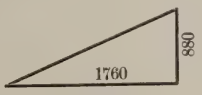
Estimating Acreage.

Rule: Twice product of dimensions in hundreds of yards=approximate number of acres (3.2 per cent small). For nearly exact arithmetical value add $\frac{1}{30}$ of above (error 0.027 per cent large). Error in approximate value will most likely be less than that due to probable error of dimensions used. Examples as figured in actual practice: Dimensions in yards, results in acres.

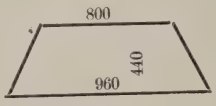
A is approximate area.
B is corrected area.
C is actual area.



17.6
26.4
704
1056
352
464.64=A
15.48
480.12=B
480.00=C



17.6
8.8
1408
1408
154.88=A
5.16
160.04=B
160.00=C



17.6
4.4
704
704
77.44=A
2.58
80.02=B
80.00=C

Proof:

43560 sq. yds.=9 acres.

Any pair of factors of 43560 taken as dimensions and expressed as so many hundred yards, gives a product of 4.3560; twice this product=8.7120 (the approximate A), or 0.288 acres less than the actual area (C) of 9 acres. Per cent of error= $\frac{0.288}{9}$ =3.2 per cent. Required correction to obtain correct arithmetical area (C)= $\frac{0.288}{8.712}$ =3.306 per cent=$\frac{1}{30}$ (nearly) of A . ·. divide A by 3, setting result one place to right and add; i. e.,

3) 8.7120
.2904
9.0024=B

and is 0.0024 acre, or $\frac{0.0024}{9}$=0.027 per cent too large.

This rule readily works backwards to determine dimensions for various areas. Divide acres (or acres$-\frac{1}{30}$) by twice one dimension to get other, both expressed in hundreds of yards. Example: Determine how much of a field 350 yards wide is required to provide 19 acres.

$$\frac{19.}{7.}=2.7=270 \text{ yards.}$$

Or if greater accuracy is desired

$$\begin{array}{r} 3\,)\,19. \\ .63 \\ \hline \frac{18.37}{7.}=2.624=2.624 \end{array}$$ yards, or approximate value may be carried out more places and reduced $\frac{1}{30}$.

[Actual length=262.7 yards.]

NOTE—For ordinary camp site work use approximate values, error is on safe side.

Raising a River Steamboat under Difficulties.

BY

Capt. JOHN N. HODGES,
Corps of Engineers.

On the morning of December 13, 1914, the steamboat *General Craighill,* owned and operated by the Government in the construction by hired labor of Dam No. 28, Ohio River, attempted to remove some barges which had been temporarily made fast to the land wall of the lock and which were in danger of going adrift, due to the rapidly rising river.

At the time in question, a large portion of the river was obstructed by a cofferdam enclosing the bear-traps, weir and abutment. This large cofferdam caused a swift current through the lock and pass with a decided set to the bank on the lock side of the river. While backing downstream with two of the barges, the steamboat was caught by the current and carried sideways across the lower guide wall. Unfortunately, at this time there was sufficient water over the lower guide wall to float the steamboat but not sufficient to permit her to clear the check posts, one of which penetrated her hull at a point directly beneath the rear end of the pilot house. The steamboat immediately settled to a horizontal position across the wall and resting on it, beam on to the current.

All plant available was rushed to her assistance, as it was hoped to keep her in a level position until the river had risen sufficiently to permit her to float off the wall. It was found, however, that the leakage resulting from the tear made by the check post was greater than the syphons available could handle, and one of the large privately owned towboats was called so as to secure some additional syphons and boiler power. This boat in coming downstream at a high rate of speed raised sufficient swell to run over the *Craighill's* deck and through her hatches, causing the boat to sink until her stern rested on the bottom of the river in rear of the guide wall. As the guide wall itself supported the boat underneath the pilot

Raising Steamboat.

house, the hull assumed a position making an angle of about 30 degrees with the horizontal.

There is nothing particularly unusual in a sunken steamboat on the Ohio River; this event being of frequent occurrence. The position assumed by the *Craighill*, however, was somewhat novel as usually it is possible to allow a steamboat about to sink to settle in shallow water on an even keel. Under normal conditions, a boat sunk in shallow water is raised by laying a barge on each side of her. Water is pumped into the barges, wire lines or chains passed under the boat and the ends made fast to the two barges. The boat is then raised by pumping out the barges. In case the depth of water in which the sunken boat is lying is considerable,

the barges are laid alongside as before, wire lines are passed under the boat and made fast to timbers which are supported at the ends by the barges. The boat is then raised by jacking up the timbers. Obviously, in addition to the above methods, a steamboat can be raised by dredges and derrick-boats, but this method is dangerous, as usually the weight to be lifted is large and requires the combined efforts of a number of pieces of floating plant. In case of the failure of a part of the hoisting machinery on one boat, an excessive load is immediately thrown on the other pieces of floating plant with disastrous results.

None of the above methods appeared applicable in the present case, for a number of reasons; among which the following may be mentioned: the inaccessibility of the *Craighill's* position, the swiftness of the current and the consequent danger to pieces of plant in laying them alongside of her; the lateness of the season and the rapid changes in river stages to be expected with the consequent danger of having other plant stranded on top of the guide wall; also the undesirability of tying up so much additional plant to the detriment of the other work.

It was decided to allow the river to fall before raising the boat. This conclusion would not have been possible in the case of the average towboat, as it is believed that the great majority of boats on the river, if placed in a position similar to that occupied by the *Craighill*, would break in two as soon as the water fell from under them, as they have not the necessary longitudinal stiffness. The *Craighill* is exceptional in this respect. She is a steel hull towboat, length over all 133 feet 2 inches, beam 28 feet ½ inch, draft 36 inches. She has three heavy longitudinal steel trusses and four bulkheads extending across the boat.

The method employed in raising the *Craighill* was as follows: As soon as the river had fallen below the top of the lower guide wall, an earth embankment was thrown up by the dredge connecting the downstream end of the lower guide wall with the river bank. The cofferdam thus formed by the lower guide wall itself, the earth embankment and the lower gate recess contained the steamboat and was unwatered by a 12-inch Morris centrifugal pump placed on the guide wall and supplied by a boiler-boat moored alongside. The suction and discharge of the pump were so arranged that the cofferdam could be unwatered and also so that water could be pumped from the river into the cofferdam with a minimum loss of time in making the change. It was found

that the check post had made a tear in the bottom plates of the steamboat about 6 feet long by 2 feet wide, and that the check post itself was firmly lodged in the plates and rested against one of the longitudinal steel trusses. Several attempts were made to cut the check post flush with the top of the wall with a cold chisel, but without success. This work was finally accomplished by the use of an oxy-acetylene cutting-torch in about ten minutes' working time.

After unwatering the cofferdam, all mud and refuse was cleaned out of the hull and the wheel removed. A timber box was built over the tear in the bottom plates and firmly held in place by jacks

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resting against the deck. A rough computation showed that the center of gravity of the steamboat was about 8 feet in rear of the guide wall and that due to the inclined position occupied by the steamboat, it would be necessary to raise the water in the cofferdam to a level that would flood part of the deck of the steamboat before buoyancy sufficient to start the stern up was obtained. As the stern was lodged in the mud, it was considered advisable to provide for considerable excess buoyancy, and with this idea in view a bulkhead of 2 by 12 inch lumber was built around the sides and stern of the boat extending up to an elevation equal to that of the top of the guide wall, so as to exclude the water from the hull and deck of the boat when the cofferdam was pumped full. This bulk-

head was laid on the outside of the engine room cabin, some additional 4 by 4 inch studs and braces added and the bulkhead well caulked so as to be water-tight.

In case the river remained at a stage below the top of the guide wall, it was the intention to pump water into the cofferdam until the boat came to an even keel and by feeding rollers at the same time between the bottom of the boat and the guide wall to pull her off the wall into the cofferdam with nigger lines, after which the earth embankment was to be removed and the boat moved into the river.

All preliminary work was completed on the evening of the 22d. During the night the river rose rapidly, and on starting work on the morning of the 23d the water was within a foot and a half of the top of the guide wall. The steamboat was in a precarious condition, as the cofferdam was practically dry and the earth embankment dangerously near to being overtopped. Work was immediately started, pumping water from the river into the cofferdam. A derrick-boat and a large number of laborers with sand bags were employed in holding the earth embankment. Fortunately, their efforts were successful and this weak point of the cofferdam was held until the river overflowed the top of the guide wall, bringing the *Craighill* to an even keel. During the night the river rose sufficiently to permit of floating the boat off the wall.

Editorial Announcements.

There is issued with this number an index of Volume VII (1915), PROFESSIONAL MEMOIRS.

It is expected that an index of Volumes I to VII will be published at an early date.

The Board of Editors will be pleased to receive photographs of works and details of works under construction or completed by the Engineer Department, accompanied by a brief notation giving title, date, and location, and other explanatory or descriptive matter. Where the subject is one which has been previously treated in PROFESSIONAL MEMOIRS, a reference to such earlier article is desirable. Payment will be made for photographs accepted for publication; the others to be transferred to the Department of Civil Engineering of the Engineer School or to be returned to the contributor, if so desired. Ordinarily, a notation of forty of fifty words will be sufficient, but longer ones will be acceptable.

Selected Articles of Engineering Interest

Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden.)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution.
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institute of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review (London).
(80) Journal, American Society of Engineering Contractors, N. Y.
(82) Journal, New England Water Works Association, Boston.
(83) National Waterways, Washington, D. C.

*Now combined under title: Concrete-Cement Age.

AERIAL WARFARE.

Aircraft in warfare; the dawn of the fourth arm. F. W. Lanchester. Parts 6-10. (11), Sept. 11, 18, Oct. 9, 16, 23, 30, Nov. 6, 1914. D.—Hydro-aeroplane float design. (10), Oct. 30, 1914. D. I.—New hangars for military uses. (28), Oct. 17, 1914. D. I.—Wireless equipment on aircraft. M. D. Manton. (28), Oct. 3, 1914. D. I.

BANK PROTECTION—RIVERS.

Flood prevention for the Lower Mississippi. T. G. Dabney. (15), Nov. 14, 1914.—Holding river in check. (40), Nov., 1914. I.

BARGES.

Canal and river barges. J. A. Bensel. (52), Oct., 1914.—Plates.

BATTLESHIPS.

The evolution of the battleship of the dreadnaught type. W. A. Dobson. (22), Oct., 1914.

BULKHEADS.

Bulkhead for Boston dry dock. (14), Nov. 26, 1914. D. I.—Bulkhead to resist unit pressure of 200 pounds. (15), Oct. 10, 1914.—Method and cost of constructing a reinforced concrete bulkhead at Hibernia Mine, N. J. (12), Sept. 30, 1914. D.

CAISSONS.

Largest pneumatic caisson job in Canada. (15), Oct. 10, 1914.—Lock-entrance caisson for the Panama Canal. (27), Nov. 14, 1914. I.

CAMP CONSTRUCTION.

Some modern methods of camp sanitation and housing. (39), Oct. 15, Nov. 15, 1914. D.

CANAL BOTTOMS.

Protecting canal bottoms from injury from screw propellers. (28), Nov. 14, 1914. D. I.

CANALS.

Design of the New York Barge canal. (14), Oct. 22, 1914.—El canal de barge. Sifon-vertedor. G. Rubio. (Boletin de ingenieros, Mexico.) Oct., 1914. D.—Action of water in the locks of the Panama Canal. H. F. Hodges. (30), Jan.-Feb., 1915. D. I.

CEMENT GUN.

Lining large steel coal tank with concrete. (15), Nov. 14, 1914. I.

CHINA.

The Huai River (China) conservancy project. Board of Engineers app. by American Red Cross. (30), Jan.-Feb., 1915. D. Maps.

COAST DEFENSE.

Coast defense in the Civil War. W. J. Buttgenbach. (43), Sept.-Oct., 1914. Map.

COFFERDAMS.

Methods of construction of the Grand Mere hyrdo-electric plant, Quebec. (14), Oct. 1, 1914. D. I.

COLORADO RIVER.

A suggested plan for developing the lower Colorado River. W. D. Smith. (14), Oct. 22, 1914. D.

CONCRETE.

Building a concrete sea wall with a special floating mixer. (14), Oct. 22, 1914. D. I.—Durability of concrete in sea-water. (11), Oct. 9, 1914.—Effect of electrolysis on the compressive strength of cement and concrete. (3), Oct. 29, 1914. D. I.—Gewapend beton. J. A. Van der Kloes. (16), Oct. 3, 1914. D. I.—Permeability tests on gravel concrete. (15), Sept. 26, 1914.—Portable concreting plant with telescopic tower. (14), Nov. 12, 1914. I.—Proof of an assumption in the theory of concrete beams. R. E. Goodwin. (21), Nov., 1914. D.—A sectional telescoping hoist-tower. (14), Nov. 12, 1914. I.—Unit steel forms for concrete floors and walls. (14), Nov. 12, 1914. D. I.—One-sided engineering contracts. (14), Nov. 12, 1914.

DAMS.

The Arrow-rock dam. C. J. Lisle. (28), Nov. 28, 1914. I.—The Calaveras dam, California; the highest earth dam. A. J. Cleary. (14), Oct. 1, 1914. D. I.—The constant-angle arch dam. D. C. Henny. (21), Oct., 1914.—Construction of dams. A. E. Walden. (82), Sept., 1914. D. I.—Floods wreck newly completed concrete dam in California. (15), Nov. 14, 1914.—A government power dam for municipal use. (14), Nov. 12, 1914. D. I.—Irrigation works in Italy. L. Luiggi. (11), Sept. 18, 1914. D.—A large hand-built earth dam, "Mammoth Reservoir," Price, Utah. J. C. Wheelon. (14), Oct. 15, 1914. I.—L'evoluzione delle grandi dighe per laghi artificiali. L. Luiggi. (Annali della societa degli ingegneri), Oct. 1, 16, 1914. D. I.—Proposed water storage on the Upper St. Maurice River, province of Quebec. J. W. Thurso. (15), Oct. 31, 1914. D.—Regulation of water levels in summit pools. (15), Aug. 1, 1914. D.—A small reinforced-concrete dam with a large upstream apron of concrete

paving. (14), Nov. 26, 1914. D. I.—Some experiences in building a small dam. J. Wilkes. (Engineering Association of the South, Proceedings.) July-Sept., 1914. I.—Steam-shovel work at Ashokan Reservoir. (15), Oct. 3, 1914. D. I.—Subterranean water and the construction of core-wall for an earth dam. (3), Sept. 24, 1914. D.—Use of hydrated lime in dam construction. (15), Nov. 14, 1914.

DERRICKS.

A 75-ton derrick for defense erection at Manila. (15), Oct. 10, 1914. D.—Test of 75-ton steel derrick. (15), Nov. 7, 1914. D. I.

DIKES.

Shore dikes on Limon Bay. (41), Oct. 21, 1914.—Steam-shovel work at Ashokan Reservoir. (15), Oct. 3, 1914. D. I.

DOCKS.

Reinforced concrete docks in North America. (3), Oct. 1, 1914.

DREDGES AND DREDGING.

Dredging progress on the Columbia River bar. (15), Oct. 3, 1914.—Hydraulic dredge to dig to depth of 100 ft. (15), Nov. 21, 1914. I.—Ladder dredges with side ladder frames. (14), Nov. 12, 1914. D. I.—Manganese-steel dipper buckets for Panama dredges. (14), Nov. 19, 1914. D. I.

EMBANKMENTS.

Questions in building the embankment of the Yale bowl. C. A. Ferry. (14), Nov. 26, 1914.

ENGINEER TROOPS.

The engineer as a factor in modern warfare. J. E. Kuhn. (15), Nov. 14, 1914. I.—The Royal Engineers of the British Army. G. Laidler. (3), Nov. 19, 1914.

ENGINEERING LITERATURE.

Filing of engineering literature. (3), Nov. 12, 1914.

EUROPEAN WAR.

Aircraft in warfare. F. W. Lanchester. (11), Sept. 11, 18, Oct. 9, 16, 23, 30, Nov. 6, 1914. D.—The automobile torpedo. (28), Oct. 3, 1914. I.—Bridge destruction in the European War. (4), Nov. 19, 1914. I.—The bridge in war time. (15), Oct. 3, 1914. I.—British and German trade and the war. (11), Oct. 9, 1914.—Care of the wounded. (28), Nov. 21, 1914. I.—El conflicto Europeo. Cronica de los acontecimientos. E. del Pozo. (Boletin de ingenieros, Mexico), Oct., 1914.—Culture and the war. Practical engineer, Lond.), Nov. 5, 1914.—Development of the military small arm. (11), Oct. 9, 1914.—Diary of the great European war of 1914. W. M. Ogg. (31), Oct., Nov., 1914.—Discurso del rey de Belgica momentos antes de la ruptura de las hostilidades. (Boletin de ingenieros, Mexico), Sept., 1914.—Engineering institutions and the war. (10), Oct. 2, 1914.—The Engineers' part in war. Editorial. (15), Nov. 14, 1914.—Engineers under war conditions. (Practical engineer, Lond.), Oct. 15, 1914.—European war notes. C. C. Gill. (46), Sept.-Oct., 1914.—Fieldmarshal Sir J. French's dispatch dated Sept. 7, 1914. (78), Oct., 1914.—French and German field artillery. (11), Oct. 9, 1914.—French forests in the war zone. S. T. Dana. (American forestry), Nov., 1914. I.—Heavy siege artillery in the European war. (Field art. journal), July-Sept., 1914. I.—How the war affects our industries. L. V. Redman. (27), Oct. 24, 1914.—Industrial arrangements for war. (Practical engineer, Lond.), Oct. 15, 1914.—Letters from the firing line. (27), Oct. 24, 1914. I.—The war in the sky. (27), Nov. 21, 1914.—Liability for damage caused by hostile aircraft. (11), Oct. 23, 1914.—Making war photographs from aeroplanes. H. Bannerman-Phillips. (27), Oct. 24, 1914. D. I.—Mechanism and the war. (Practical engineer, Lond.), Oct. 29, 1914.—The motor car and the war. (Prac. engineer, Lond.), Oct. 22, 1914.—Motor vehicles and transportation in the European war. R. W. Hutchinson. (13), Oct., Nov., 1914. I.—Motorcycles in the war. (27), Nov. 28, 1914.—Naval fights in the Pacific and Indian oceans. (27), Nov. 21, 1914. I.—New flashlight apparatus for army use. (27), Oct. 24, 1914. I.—Nocturnal warfare. The automobile-mounted searchlight of the French army. (28), Oct. 3, 1914. I.—Patents and trade-marks and the war.

(10), Oct. 2, 1914.—Pictures from the front. (27), Nov. 14, 1914. I.—Prices as affected by the European war. A. Del Mar. (13), Oct., 1914.—Recruiting and the manufacture of war munitions. (11), Sept. 11, 1914.—Scrap metal from battlefields. (Pract. engineer, Lond.), Oct. 22, 1914.—Shipping trade and the war. (10), Oct. 2, 1914.—Strategic moves of the war. (27), Oct. 17, 24, Nov. 14, 21, 28, 1914. D. I.—Trade competition with our enemies. (10), Oct. 2, 1914.—Unpatriotic British workmen. (11), Oct. 9, 1914.—Vae victis. C. Almazan. (Boletin de ingenieros, Mexico, Sept., 1914.—War and engineering trade abroad. (3), Sept. 24, 1914.—War and the British engineer. (10), Sept. 11, 18, 25, Oct. 2, 16, 1914.—War engineering. (15), Sept. 26, 1914.—The war in Europe. (27), Oct. 10, 1914.—War items. (10), Oct. 9, 30, 1914.—Wars dislocation of supply and demand. (11), Oct. 30, 1914.—Wireless equipment on air-craft. M. D. Manton. (28), Oct. 3, 1914. D. I.

EXCAVATION AND EXCAVATORS.

Excavating machine walks to its work in Iowa town. (15), Nov. 14, 1914. I.—Novel method of trench excavation. J. C. Lathrop. (14), Nov. 19, 1914. I.

EXHIBITIONS.

Structural features of the 150-ft. steel framed dome of the palace of horticulture, Panama-Pacific International Exposition, San Francisco, Cal. A. W. Earl. (12), Sept. 30, 1914. D. I.

EXPLOSIVES.

A new explosive. (27), Nov. 14, 1914.—The various materials employed and methods of using. (28), Oct. 17, 1914. I.

FLOODS.

Concrete retaining wall for flood protection at Schenectady. (14), Oct. 8, 1914. I.—Control of Mississippi floods. W. H. Schuerman. (Engineering Association of the South. Proceed.), April-June, July-Sept., 1914. D.—A flood from a 7-in. rainfall at Kansas City. (14), Sept. 24, 1914.—Flood prevention for the Lower Mississippi. T. G. Dabney. (15), Nov. 14, 1914.—Flood prevention reservoirs in Bohemia. K. C. Grant. (14), Oct. 15, 1914. D. I.—A flood-proof road. (27), Nov. 14, 1914.—Floods wreck newly completed concrete dam in California. (15), Nov. 14, 1914.—Kansas City flooded. E. B. Murray. (15), Sept. 26, 1914. I.—Protection of Watervliet, N. Y., against storm waters. F. J. Keis. (14), Nov. 26, 1914. D. I.—Regulating a troublesome stream at Watervliet. (15), Nov. 21, 1914. I.—The Huai River (China) conservancy project. Board of Engineers app. by Am. Red Cross. (30), Jan.-Feb., 1915. D. Maps.

FORTIFICATION, FIELD.

Protection for the fighting line of an army. L. S. Roudiez. (27), Oct. 3, 1914. D. I.

FORTIFICATION, PERMANENT.

Passing of permanent field fortifications. (27), Oct. 24, 1914.

FOUNDATIONS.

Percolation and upward pressure of water. W. A. Mitchell. (30), Jan.-Feb., 1915. D.

HARBORS.

Australian ports in relation to modern ships and shipping. W. E. Adams. (11), Sept. 18, 1914.—Federal and local duties in port development. W. M. Black. (15), Sept. 26, 1914.—State control over ports and water-fronts. B. F. Cresson. (15), Sept. 26, 1914.—Toronto harbor and water-front development. (3), Oct. 1, 1914. D. I.

HYDRAULICS.

Diaphragm method of measuring the flow of water in open channels of uniform cross section. (12), Oct. 28, 1914. D.—Action of water in the locks of the Panama Canal. H. F. Hodges. (30), Jan.-Feb., 1915. D. I.

INTRENCHING TOOLS.

Protection for the fighting line of an army. L. S. Roudiez. (27), Oct. 3, 1914. D. I.

IRRIGATION.

Irrigation works in Italy. L. Luiggi. (11), Sept. 18, 1914. D.—The twenty-first International Irrigation Congress. F. H. Newell. (14), Oct. 22, 1914.

JACKS.

A hydraulic jack for emergency service. (14), Nov. 19, 1914.

LAND-DRAINAGE.

Draining district 9, Mississippi County, Ark. L. L. Hidinger. (15), Oct. 24, 1914. D.

LANDSLIDES.

Clearing the slide in the Panama Canal. (14), Nov. 12, 1914. I.—The slide that blocked Panama traffic. (15), Nov. 7, 1914.

LEVEES.

Building levees with the hydraulic dredge. J. M. Allen. (14), Oct. 29, 1914. D. I.—Draining district 9, Mississippi County, Ark. L. L. Hidinger. (15), Oct. 24, 1914. D.—Experimental levees with concrete paving and sheet-pile cut-off, Mississippi River. C. O. Sherrill. (14), Oct. 8, 1914. D. I.—A flood from a rainfall at Kansas City. (14), Sept. 24, 1914.—Indiana levee district. (15), Aug. 1, 1914. D.—Kansas City flooded. E. B. Murray. (15), Sept. 26, 1914. I.

LEVELING.

Rod for both stadia and direct leveling. W. S. Nichols. (14), Oct. 29, 1914. I.

LOCKS AND LOCK GATES.

Floating lock-gate lifter for Trent Canal, Canada. (3), Oct. 1, 1914. I. (14), Nov. 19, 1914. I.—Lock at Sault Ste. Marie in operation. (15), Oct. 31, 1914. I.—Action of water in the locks of the Panama Canal. H. F. Hodges. (30), Jan.-Feb., 1915. D. I.

LOG RAFTS.

Ocean-going log rafts. J. F. Springer. (27), Nov. 14, 1914. I.

MATTRESSES.

Novel cross-section instrument for valuation work. (15), Oct. 3, 1914. I.

MILITARY ENGINEERING.

The engineer as a factor in modern warfare. J. E. Kuhn. (15), Nov. 14, 1914. I.—Military engineering training for the engineering student. (14), Oct. 22, 1914.

MILITARY TRANSPORTATION.

Military road-transport. (10), Sept. 18, 1914.—Motor vehicles and transportation in the European war. R. W. Hutchnson. (13), Oct., 1914. I.

MISSISSIPPI RIVER.

How the ancients would have controlled the Mississippi. C. E. Henderson. (15), Aug. 1, 1914.—The same. W. Willcocks. (42), June, 1914. D.

MOTOR CARS AND TRUCKS.

The greatest practical test for the motor truck. (3), Nov. 19, 1914.—Electric vehicles. (10), Oct. 23, 1914. I.—A few motor-truck novelties for contractors. (14), Sept. 24, 1914. D. I.—Motor vehicles and transportation in the European war. R. W. Hutchinson. (13), Oct., Nov., 1914. I.

ORDNANCE.

The enemy's guns. (10), Sept. 25, 1914. D. I.—Germany's big siege guns. (27), Nov. 14, 1914.—The German siege howitzers. (31), Oct., 1914. I.—Heavy siege guns. (10), Oct. 23, 1914. I.

PANAMA CANAL.

Government operations at the Panama Canal. (14), Oct. 29, 1914.—The Panama Canal and the transcontinental railways. (14), Nov. 19, 1914.—The year's work on the Panama Canal. (15), Nov. 21, 1914. I.

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Selected Articles of Engineering Interest

Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden.)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
(50) Electrical Review (Chicago).
(51) Journal, Military Service Institution.
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institute of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review (London).
(80) Journal, American Society of Engineering Contractors, N. Y.
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(83) National Waterways, Washington, D. C.

*Now combined under title: Concrete-Cement Age.

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Phenomenal run of precise leveling in northwest. (15), Nov. 28, 1914.—Recent improvements in leveling instruments. D. D. Scott. (70), v. 194, 1914.

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A Dutch pile driver. (14), Dec. 3, 1914. I.—Pile driver with traversing leads for driving trestle bents. (14), Jan. 7, 1915. D. I.

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Piling used in Honolulu harbor. (12), Dec. 9, 1914.—Unique features in a municipal pier. (40), Jan., 1915.

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Rotary snow-ploughs. H. H. Vaughan. (70), v. 195, 1914.

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The strength of wire rope. (46), Nov.-Dec., 1914.

Selected Articles of Engineering Interest

(Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School, for PROFESSIONAL MEMOIRS, v. 7, No. 33, May-June, 1915.)

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden.)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
(47) Revue du Genie Militaire (Paris).
(48) La Technique Moderne (Paris).
(49) Electrical World.
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(51) Journal, Military Service Institution.
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institute of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review (London).
(80) Journal, American Society of Engineering Contractors, N. Y.
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(83) National Waterways, Washington, D. C.

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The United States an undefended treasure land. (27), Feb. 6, 13, 1915. I.

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The engineer's interest in deep water-ways, with special reference to the Mississippi River and its tributaries. H. N. Wagner. (3), Jan. 28, 1915.—New coast line of Northern Pacific Railway, near Tacoma. (14), March 25, 1915. D. I.—Protecting the banks of Red River against flood damage. (39), Feb. 15, 1915. I.

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Would operate self-propelled barges on the Mississippi. (15), March 6, 1915.

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Aerial ropeways. H. Davies. (10), Jan. 15, Feb. 5, 12, 19; March 5, 1915. D. I.

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American Concrete Institute's tests on concrete columns. (14), March 18, 1915. D.—British Portland cement making machinery. (10), Feb. 12, 19, 26; March 12, 1915. D. I.—Building concrete scows is made a commercial success. A. W. King. (40), April, 1915. I.—Concrete briquettes subjected to cold weather tests. (15), Feb. 27, 1915. D.—Covering protects large penstocks from freezing. H. G. Huber. (15), Feb. 27, 1915. D. I.—Effect of bombardment on concrete. (28), Feb. 6, 1915.—Mechanical disintegration of defective concretes. N. C. Johnson. (15), Feb. 6, 1915. I. —Methods and costs of constructing concrete lining for twenty irrigation canals. (12), Feb. 10, 1915. D. I.—Methods and costs of lining irrigation canals with concrete. (39), April 1, 1915. D.—Microscope as an aid in proportioning concrete for strength. N. C. Johnson. (15), Feb. 13, 1915. I.—Microscope as a check on construction. N. C. Johnson. (15), Feb. 27, 1915. I.—Microscope shows importance of mixing as a factor in making strong concrete. N. C. Johnson. (15), March 6, 1915. D. I.—A movable concrete tower home-made skip. (14), April 1, 1915. I.—Some experiences with concrete in the republic of Panama. A. P. Crary. (14), Feb. 4, 1915. D. I.

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The corrosion of iron. L. C. Wilson. (13), Feb., March, April, 1915.

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Hoists for reinforcing-steel. H. Da Camara. (14), Feb. 25, 1915. D.

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Portable derrick for builders and contractors. (14), March 11, 1915.—Small derrick erects large pipe at fast rate. (15), March 27, 1915. I.

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The Little River drainage district. (14), March 18, 1915. D.

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The assessment of damage to maritime works. E. Latham. (11), Jan. 8, 1915. D. I.—Cut-off wall and rock grouting at the Milton Reservoir embankment. (14), March 11, 1915. D. I.

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Advance notes on the Panama-Pacific Exposition. F. H. Colvin. (2), Feb. 11, 1915. I.—Engineering problems of the Panama-Pacific Exposition. A. H. Markwart. (14), Feb. 18, 1915. D. I.—The great International Panama-Pacific Exposition. (27), Feb. 27, 1915. I.—Illumination of Panama-Pacific Exposition. G. L. Bayley. (49), Feb. 13, 1915. I.—L'Exposition internationale de San Francisco. P. Calfas. (Le Genie Civil), Feb. 20, 1915. I.—A new San Francisco has risen from the ruins of 1906. (15), Feb. 20, 1915. D. I.—The opening of the Panama-Pacific International Exposition. (2), March 11, 1915.—The Panama-Pacific International Exposition, San Francisco, Cal., Feb. 20, to Dec. 4, 1915. (Power), Feb. 23, 1915. D. I.—San Francisco's notable engineering works. (15), Feb. 20, 1915. I.—Some spectacular illuminating effects at the Pan-Pacific Exposition. (27), Feb. 20, 1915. D. I.—Venti-

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A novel plan for stopping a landslide at Mount Vernon. N. H. Darton. (14), Feb. 25, 1915. I.

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An aerial monorail. (27), March 27, 1915. D.—Le chemin de fer electrique monorail aerien de Genes, Italie. (Le Genie Civil), Feb. 27, 1915. D. I.

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Cheap and rapid gun-making. J. Perry. (11), March 12, 1915.—Forging the inner tube of a German 16½-inch siege gun. (27), March 6, 1915.—From smooth-bore to rifled cannon. (2), April 8, 1915. I.—Gun-primers and detonators. (11), Jan. 15, 1915.—The Lewis light magazine gun. (28), Feb. 6, 1915. D. I.—Long-range guns in forts. F. A. de Peyster. (27), March 6, 1915. Machining and erecting 12-inch mortar carriages. G. T. Huxford. (2), Jan. 28, 1915. D. I.—Manufacturing of cartridge-cases. (2), Feb. 4, 1915. D. I.—Maxim machine gun and its construction. (28), Feb. 6, 1915. D. I.—Les mitrailleuses automatiques. A. Reynier. (Le Genie Civil), Jan. 2, 9, 1915. D. I.—The new French "75" field gun. (28), Feb. 6, 1915.—Sir Hiram Maxim on the Maxim gun in war. H. S. Maxim. (28), March 27, 1915.—The "soixante quinze." (10), Jan. 22, 1915. D. I.—Some details and methods of constructing modern weapons. (28), Feb. 20, 1915. D.—Some machine operations in making guns. (2), Feb. 4, 1915. I.—Testing and special fixtures for gun parts. (2), Feb. 11, 1915. I.

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The Panama Canal and the ports of the Pacific. A. J. Quigley. (13), April, 1915. I.—Via Panama to Pacific ports. A. J. Quigley. (13), April, 1915. I.

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Administration building for the canal. (40), March, 1915. I.—The floating crane failure at the Panama Canal. (11), Feb. 26, 1915. D.—A great relief map of the Panama Canal. (14), Feb. 18, 1915. I.—Mechanical and electrical features of the Panama Canal. (50), Feb. 27, 1915.—The Panama Canal and the ports of the Pacific. A. J. Quigley. (13), March, 1915. D. I.—Panama Canal in miniature operates at Exposition. (15), April 3, 1915. I.—Panama Canal open to large ocean liners. (27), March 13, 1915. I.—Repairs under way to breakwater at Panama damaged by storm. (15), April 3, 1915.—Swinging pontoon bridge carries railroads across Panama Canal. (15) Feb. 27, 1915. D.—The towing locomotives for the Panama Canal. C. W. Larson. (General Electric Review), Feb., 1915. D. I.—Via Panama to Pacific ports. A. J. Quigley. (13), April, 1915. I.—The working model of the Panama Canal at the Exposition. (14), Feb. 18, 1915.

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PIERS.

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Water-proofing of solid steel-floor railroad bridges. S. T. Wagner. (21), Dec., 1914.

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The flow of sediment in the Mississippi River and its influence on the slope and discharge. C. M. Townsend. (30), May-June, 1915. D.

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The improvement of the river Clyde and harbour of Glasgow, 1873-1914. T. Mason. (11), March 12, 1915.—Improvement of the river Trent. (10), Feb. 5, 1915. D.—Snag boats on flood rivers. D. A. Willey. (28), March 6, 1915. I.

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Autotraction drill rig and hollow piston drill. (12), Feb. 24, 1915. I.—The Fort Wayne electric rock drill. C. Jackson. (General Electric Review), April, 1915. I.

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The building of concrete scows is made a commercial success. A. W. King. (40), April, 1915. I.

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Assessment of damage to maritime works. E. Latham. (11), Jan. 8, 22, 1915. D. I.—Improvement of San Francisco's waterfront. J. Newman. (14), Feb. 18, 1915. D. I.—New coast line of Northern Pacific Railway, near Tacoma. (14), March 25, 1915. D. I.

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Advance in sewage disposal. G. W. Fuller. (15), Jan. 2, 1915.—Buffalo sewage disposal and water-supply in relation to the pollution of the Great Lakes. (14), Jan. 7, 1915.—Sewage disposal plant at Aberdeen, South Dakota. W. G. Potter. (18), Oct., 1914. D. I.—Drainage of the city of Cairo, Egypt. (In: A recent trip to Italy, etc.) E. L. Corthell. (30), May-June, 1915. D. I. Maps.

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A unique method of sinking shafts in soft ground. L. T. Emory. (14), Feb. 25, 1915. I.—Winding with counter-weight in shaft-sinking. (70), v. 196, 1914.

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Concrete "bleachers" protect Massachusetts shores. (15), April 3, 1915. D. I.—New coast line of Northern Pacific Railway near Tacoma. (14), March 25, 1915. D. I.

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The flow of sediment in the Mississippi River and its influence on the slope and

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Snag boats on flood rivers. D. A. Willey. (28), March 6, 1915. I.

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Motor snow-plow for streets. (14), Jan. 28, 1915. I.

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Pelle a vapeur pour l'execution rapide des terrassements. (Le Genie Civil), Jan. 9, 1915. D. I.—Safety first on steam shovels. (15), Jan. 30, 1915. I.

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Steel structural engineering. E. Marburg. (15), Jan. 2, 1915.

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The United States an undefended treasure land. (28), Feb. 6, 1915. I.

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Selected Articles of Engineering Interest

(Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School, for PROFESSIONAL MEMOIRS, v. 7, No. 34, July-August, 1915.)

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden.)
(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
(39) The Contractor.
(40) Cement Era.
(41) Canal Record (Ancon, C. Z.).
(42) Proceedings, Engineers' Society of Western Pennsylvania.
(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
(46) United States Naval Institute. Proceedings.
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(48) La Technique Moderne (Paris).
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(51) Journal, Military Service Institution.
(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
(65) Journal, Engineers' Society of Pennsylvania. (Harrisburg, Pa.)
(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institute of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review (London).
(80) Journal, American Society of Engineering Contractors, N. Y.
(82) Journal, New England Water Works Association, Boston.
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The improvement of the river Clyde and harbour of Glasgow, 1873-1914. T. Mason. (10), Mar. 19, 1915.

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HYDRO-ELECTRIC PLANTS.

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LANDSLIDES.

Landslide on the Kanawha and Michigan Railway. (14), April 15, 1915. I.—The slides in the Culebra cut on the Panama Canal. (14), April 22, 1915.

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Construction details of the Panama Canal lock gates. R. A. Pendergrass. (42), Nov., 1914. I. Plates.—Contract methods and equipment for a typical Ohio River dam. K. H. Shriver. (14), April 29, 1915. D. I.—Lock gate lifter for Trent Canal. (Marine Engr'g of Canada), Sept., 1914. I.—The new government ship lock in Black Rock Harbour. (Concrete & Constructional Engr'g), Oct., 1914. D. I.—Vertically framed mitering gates. M. Elliott. (30), July-Aug., 1915.

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The European infantryman's rifle. E. C. Crossman. (27), May 1, 1915. I.

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Selected Articles of Engineering Interest

(Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School, for PROFESSIONAL MEMOIRS, v. 7, No. 35, September-October, 1915.)

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
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(37) Revue d'Artillerie (Paris).
(38) Kriegstechnische Zeitschrift (Berlin).
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(40) Cement Era.
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(42) Proceedings, Engineers' Society of Western Pennsylvania.
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(44) Transactions, Society of Engineers (London).
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(70) Minutes of Proceedings, Institute of Civil Engineers, London.
(72) Institute of Engineers and Shipbuilders in Scotland. Transactions.
(78) The Army Review (London).
(80) Journal, American Society of Engineering Contractors, N. Y.
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Selected Articles of Engineering Interest

(Compiled by Mr. Henry E. Haferkorn, Librarian, Engineer School, for PROFESSIONAL MEMOIRS, v. 7, No. 36, November-December, 1915.)

In the lists of selected articles published, the publication is referred to by the number preceding its title in the following list. The following abbreviations will be used: I, for illustrated; D, for diagrams.

(1) Annales des Ponts et Chaussees.
(2) American Machinist.
(3) Canadian Engineer.
(4) Canadian Soc. of Engineers. Trans.
(5) Cassier's Magazine.
(6) Cement.
(7) Cement Age.*
(8) Cornell Civil Engineer.
(9) Electrical Review (London).
(10) Engineer (London).
(11) Engineering (London).
(12) Engineering & Contracting.
(13) Engineering Magazine.
(14) Engineering News.
(15) Engineering Record.
(16) De Ingenieur (Hague, Holland).
(17) Journal of American Society of Mechanical Engineers.
(18) Journal of Western Society of Engineers.
(19) Journal of Franklin Institute.
(20) Journal of Royal United Service Institution (London).
(21) Proceedings, American Society of Civil Engineers.
(22) Proceedings, Engineers' Club of Philadelphia.
(23) Municipal Engineering.
(24) Municipal Journal and Engineer.
(25) Railway Age Gazette.
(26) Revue Generale des Chemins de Fer (Paris).
(27) Scientific American.
(28) Scientific American Supplement.
(29) Transactions, American Society of Civil Engineers.
(30) Professional Memoirs, Corps of Engineers.
(31) Journal of the Royal Artillery (Woolwich, England).
(32) Royal Engineers' Journal (Chatham, England).
(33) Proceedings Brooklyn Engineers' Club.
(34) Concrete.*
(35) Bulletin de la Presse et de la Bibliographie militaires (Brussels).
(36) Internationale Revue ueber die gesamten Armeen und Flotten (German and French). (Dresden.)
(37) Revue d'Artillerie (Paris).
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(39) The Contractor.
(40) Cement Era.
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(43) Journal, United States Artillery.
(44) Transactions, Society of Engineers (London).
(45) Journal, Association of Engineering Societies.
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(52) Barge Canal Bulletin.
(62) Connecticut Society of Civil Engineers. Papers and transactions.
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An acreage rule of thumb. G. E. Gibson. (30), Nov.-Dec., 1915. D.

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Kiel, the great German naval Base. A. W. Metcalf. (28), Oct. 9, 1915.

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Quartermaster and submarine wharf at Key West, Fla. J. R. Slattery. (30), Nov.-Dec., 1915. D.

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Discussion: Vertically framed mitering lock gates. Harry F. Hodges. (30), Nov.-Dec., 1915.—Interesting method of erecting lock gates. (3), Oct. 7, 1915. I.—Mitering lock gate at Keokuk presenting novel features. B. H. Parsons. (15), Sept. 18 and Oct. 9, 1915.

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Double-ender military locomotives. H. T. Walker. (27), Aug. 14, 1915. I.—South African railways and the war. (11), Sept. 3, 1915. I.

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Mississippi River. J. W. Woermann. (45), Sept., 1915.

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Compilation of experiences and conclusions of eminent authorities as to the utility of outlets in reducing the height of floods, citing all opinions found, whether for or against. J. Aug. Ockerson. (30), Nov.-Dec., 1915.

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Balboa coaling plant. (11), Aug. 13, 1915.—Panama Canal. (11), Sept. 3, 1915. I. D.—Panama Canal and the West Indies. F. E. Whitton. (31), June, 1915.—Panama Canal, Balboa terminal. (11), July 23, 1915. D.—Unique bridge across Panama Canal. (3). Sept. 2, 1915.

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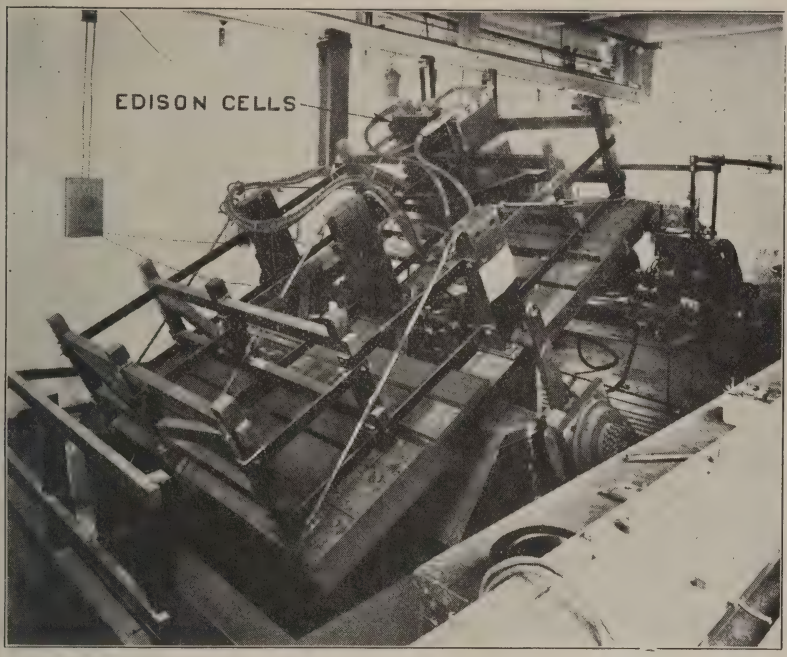

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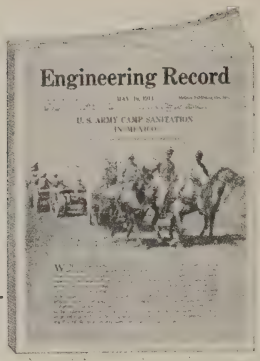

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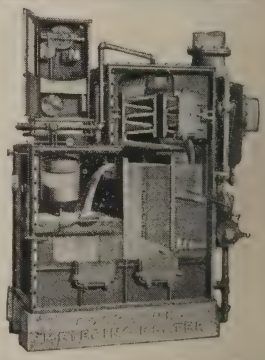

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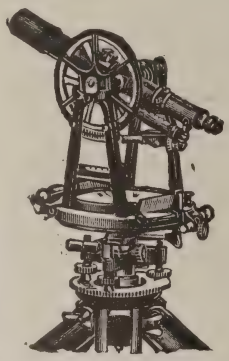

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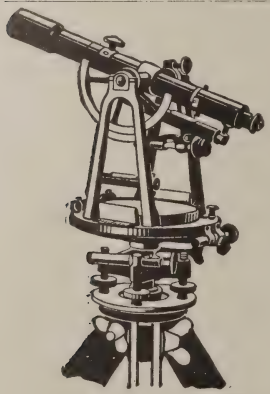

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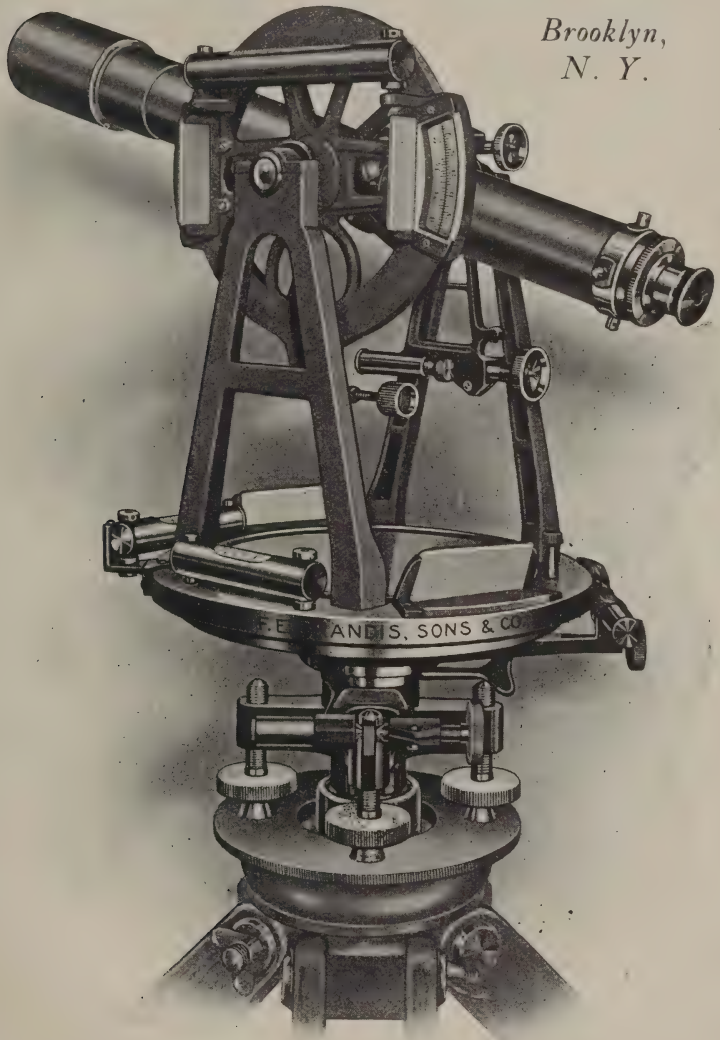

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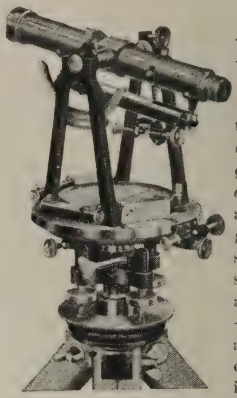

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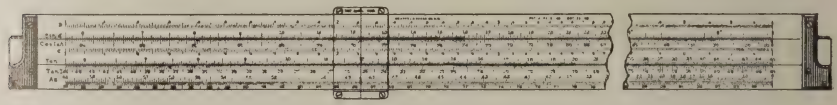

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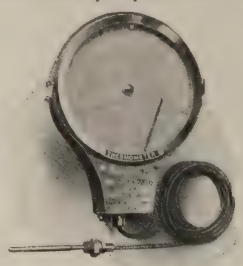

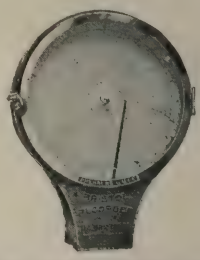

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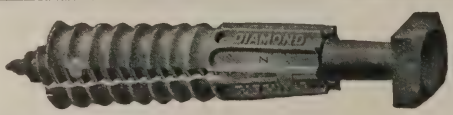

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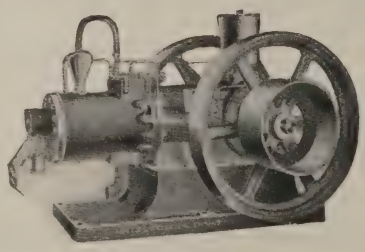

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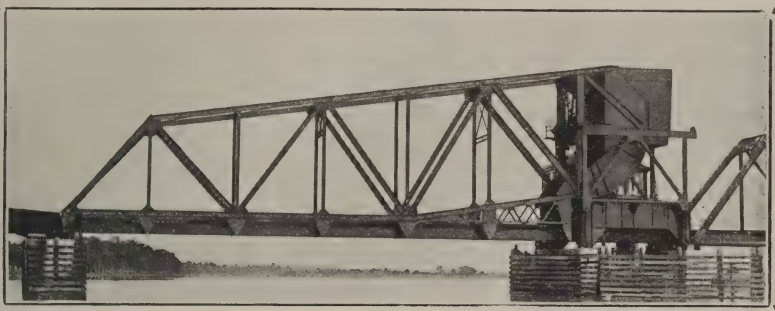

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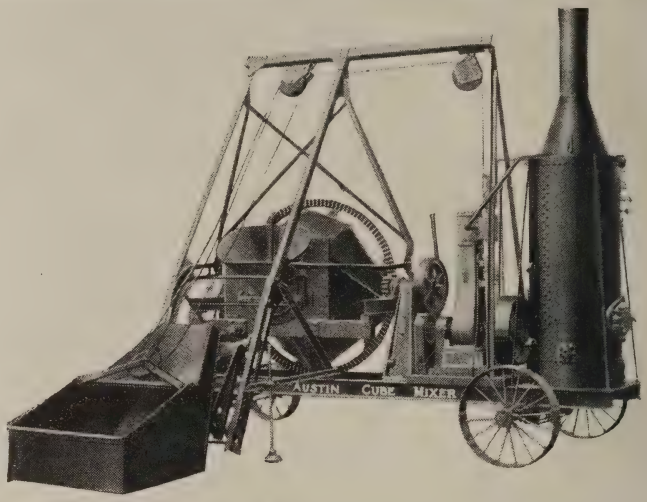

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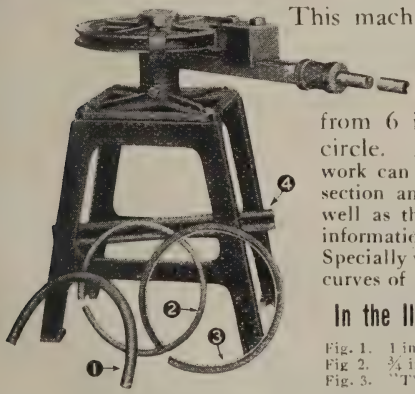

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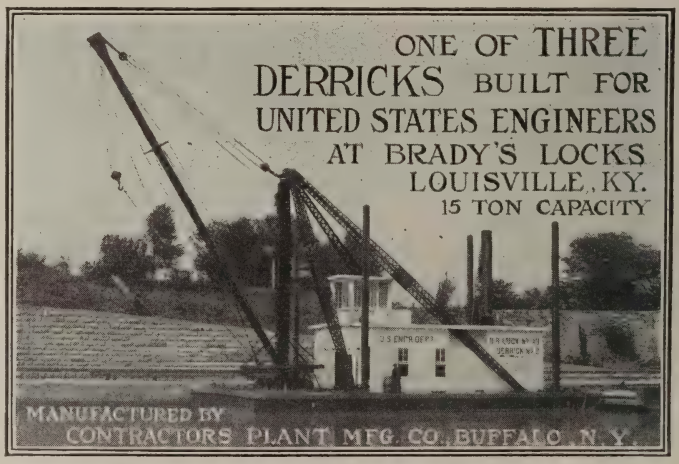

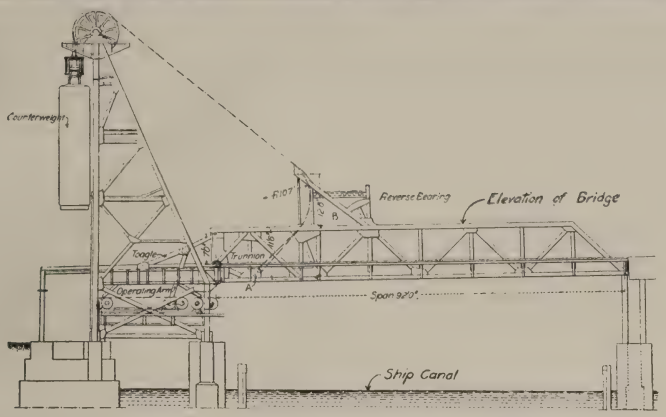

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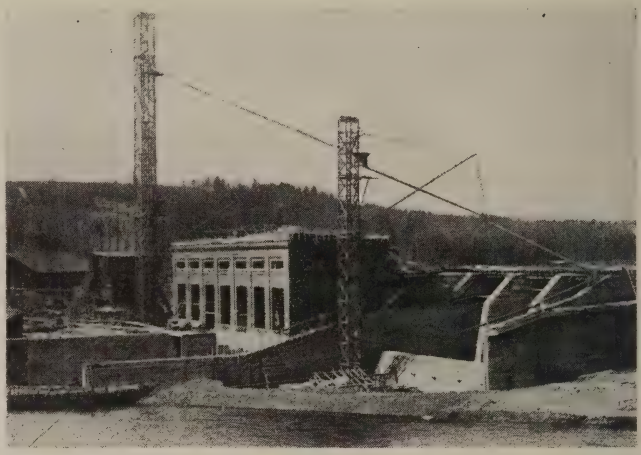

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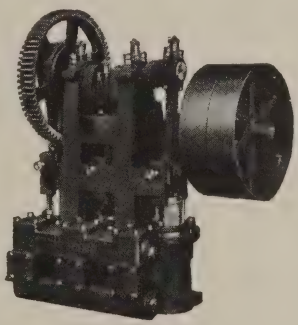

Fig. 50. Deming Belt Driven Triplex Power Pump

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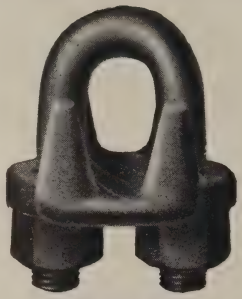

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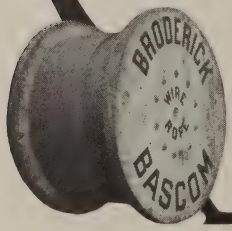

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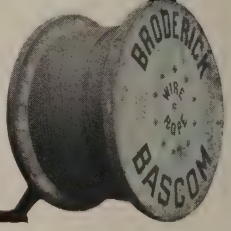

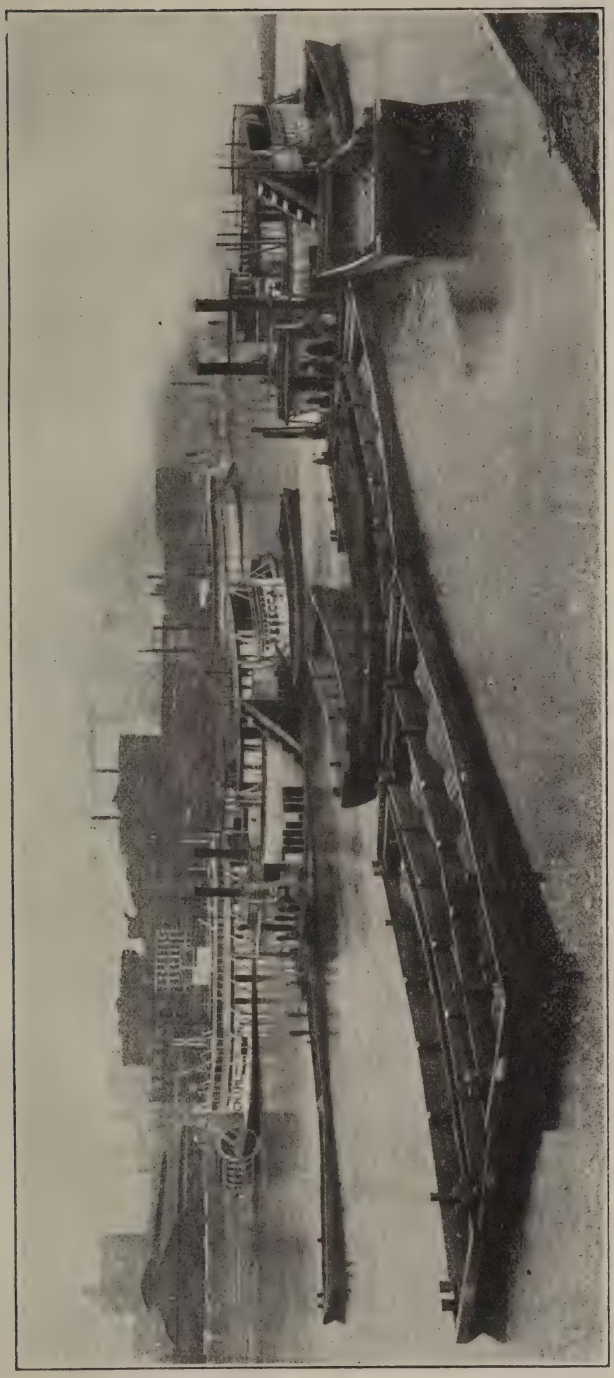

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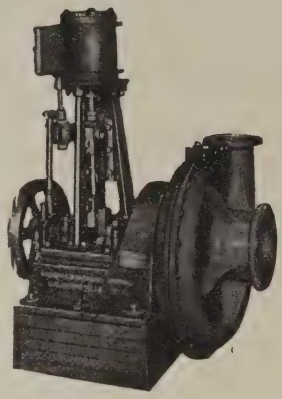

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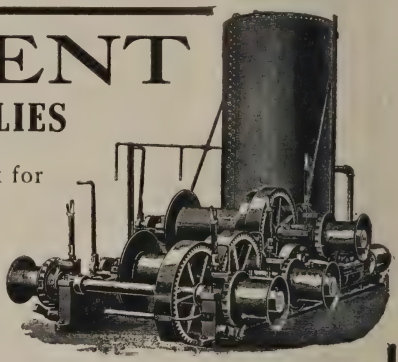

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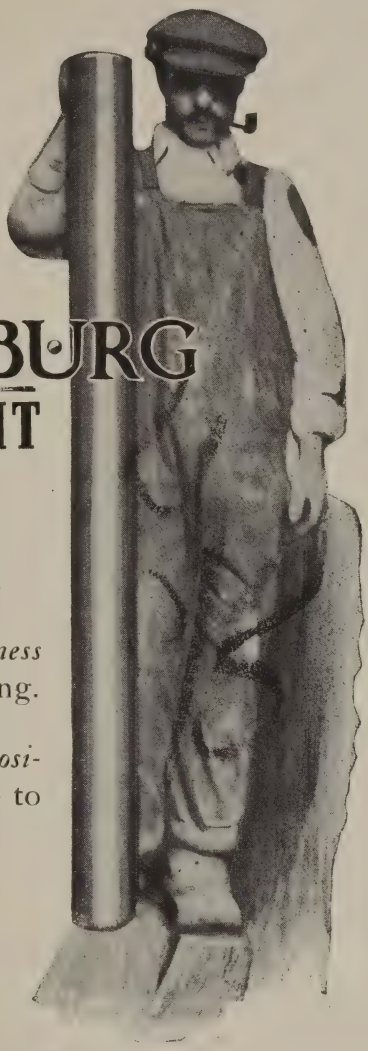

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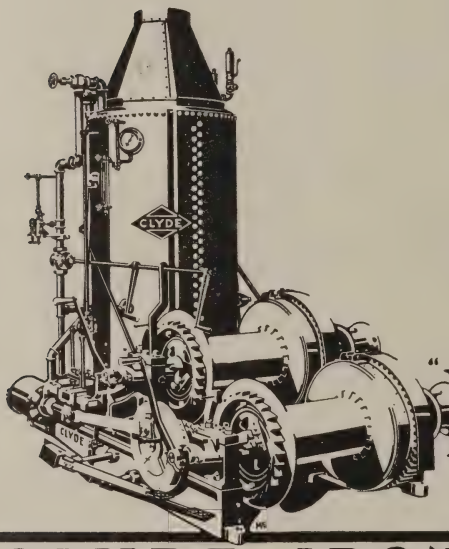

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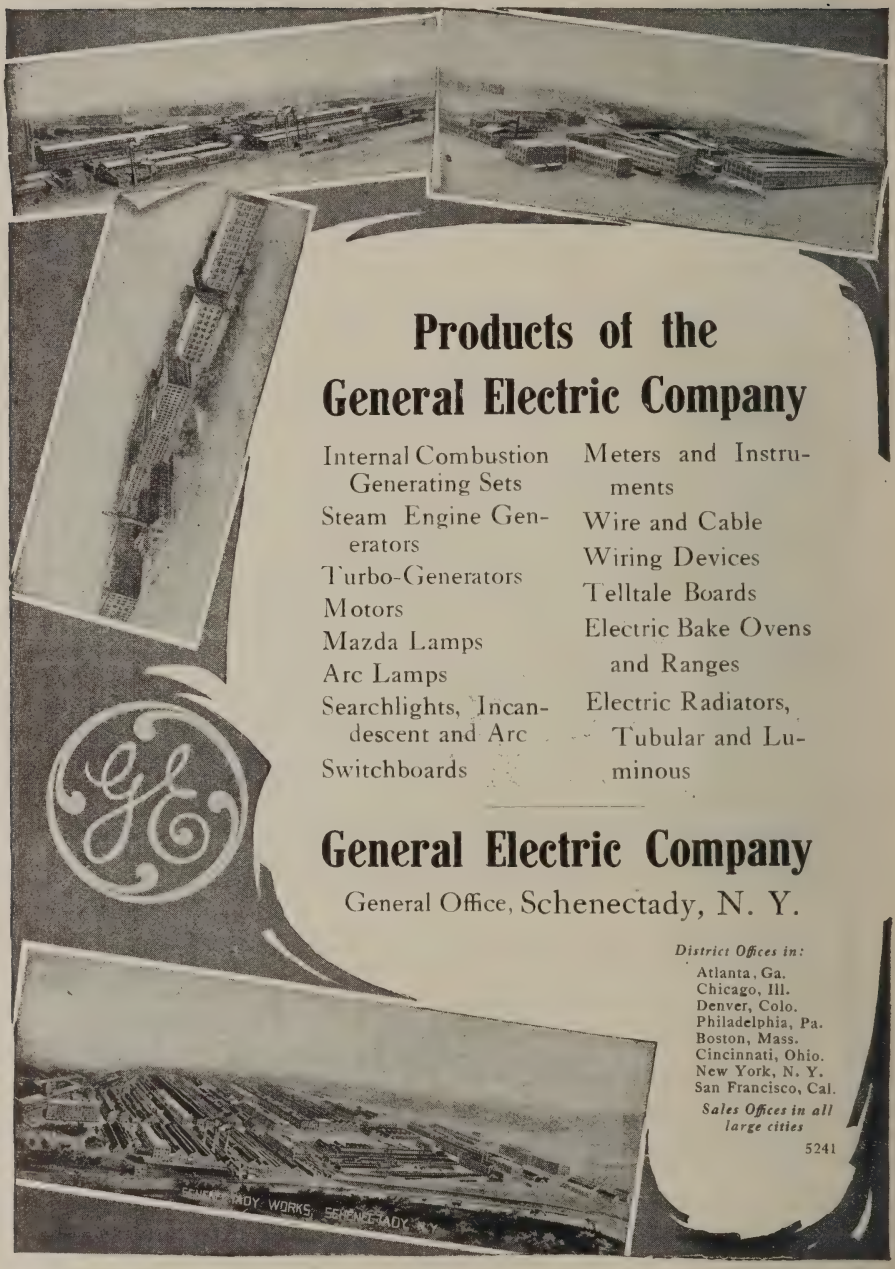

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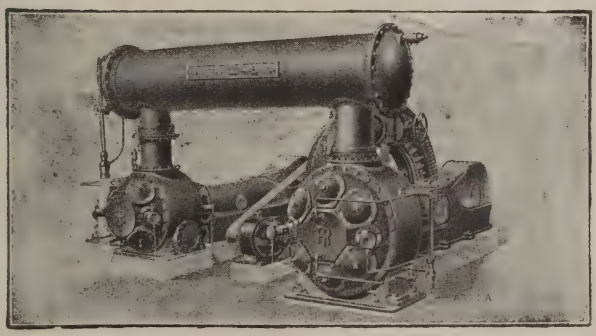

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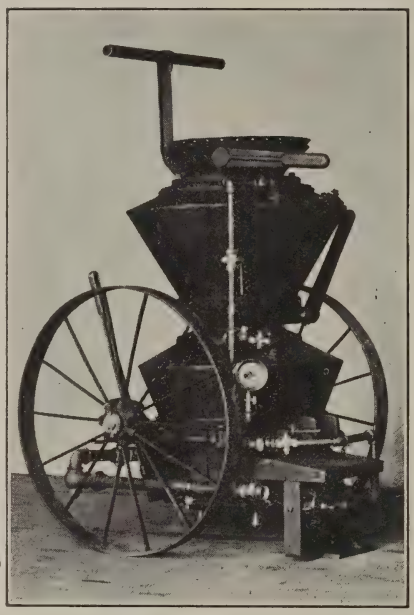

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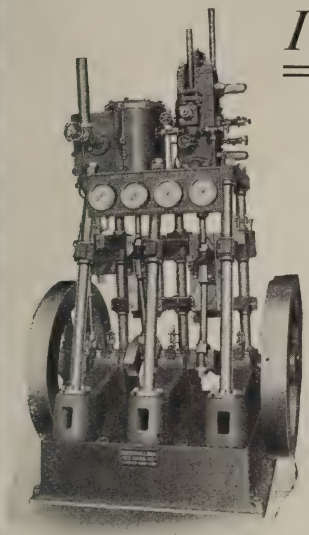

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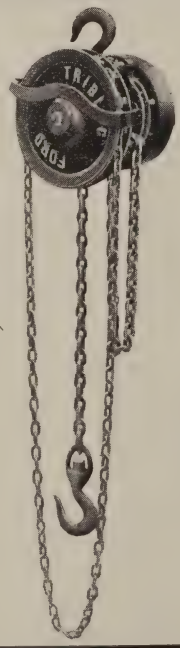

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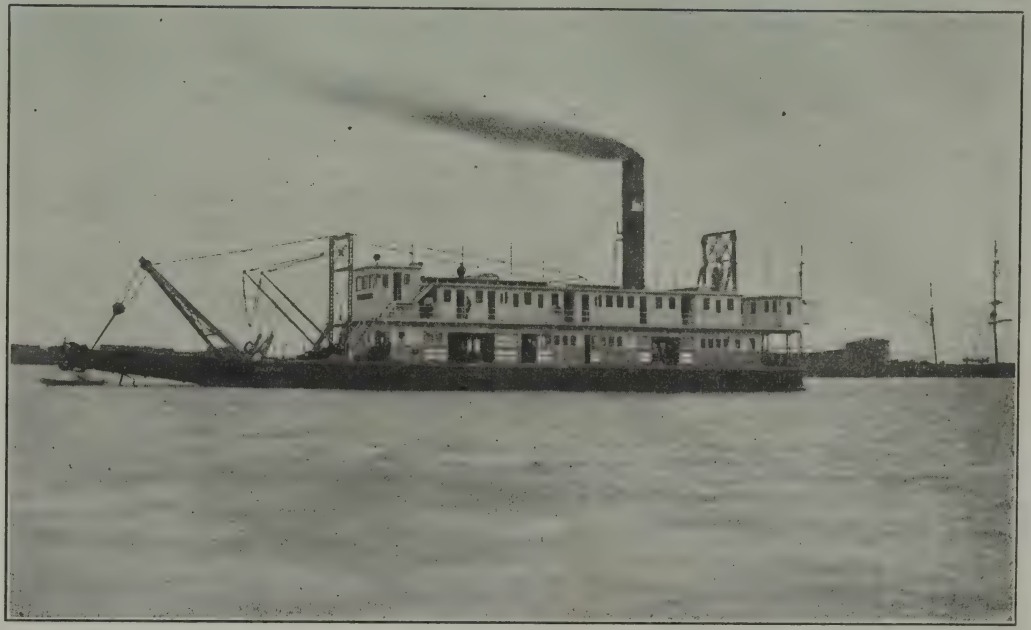

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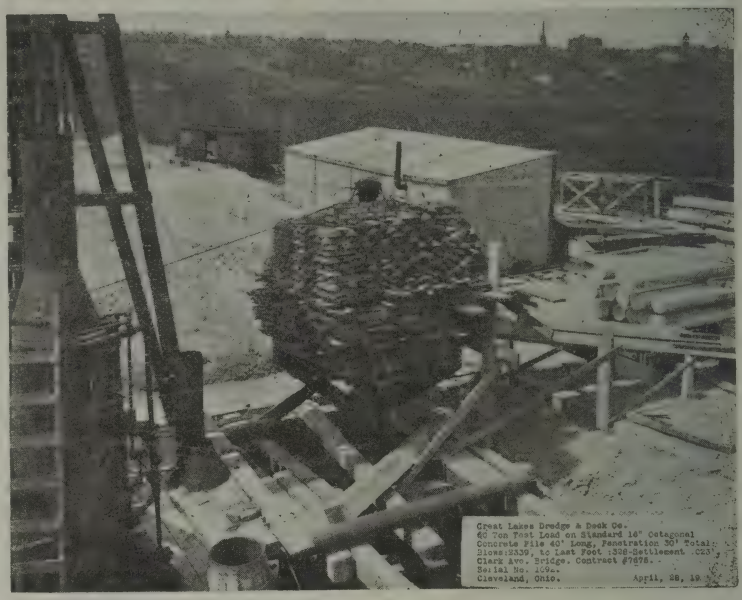
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